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THE OPERATION OF MULTIPLE RESERVOIR SYSTEMS

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THE OPERATION OF MULTIPLE RESERVOIR SYSTEMS

Z. Kaczmarek and J. Kindler
Editors

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FOREWORD

The International Institute for Applied Systems Analysis is a nongovernmental, multidisciplinary, international research institution whose goal is to bring together scientists from around the world to work on problems of common interest.

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PREFACE

With the increasing number of multiple storage reservoirs being built, systems analysis is gaining importance in planning and operating such projects. It is rather seldom, however, that their operation is given sufficient emphasis at the numerous conferences on water resource systems. Therefore, the Resources and Environment Area of the International Institute for Applied Systems Analysis (IIASA), the Committee on Water Resources of the Polish Academy of Sciences (KGW-PAN), and the Institute of Meteorology and Water Management (IGMW), Warsaw, Poland, decided to cooperate in the organization of the international workshop on the Operation of Multiple Reservoir Systems.

The purpose of the workshop was to discuss, compare and evaluate various methods of operating, and determining operating policies for multi-purpose, multiple reservoir systems. While total reservoir capacities are known, the allocation of various storage volume zones within the reservoirs to various purposes, such as water supply or flood control, may vary in time and be dependent on reservoir operating policy. In the guidelines for preparation of the workshop papers, it was stressed that discussion should cover reservoir operation for all possible hydrological situations, i.e., in periods of flood, normal flow, and drought conditions. The importance of procedures used to determine when an extreme situation, (such as drought), begins and ends, was emphasized, especially with respect to possible changes in reservoir operation. The workshop participants were requested to discuss how operating policies might differ, depending on whether the reservoirs are in series or are parallel. Also, they were requested to distinguish between operation of "large" over-year storage reservoirs and "small" within-year storage reservoirs. If short-term hydrologic forecasting and real-time control models were to be used, the workshop discussion would consider how these could be used together with long-term operating policies.

The workshop was held in Jodłowy Dwor, Poland, from May 28 to June 1, 1979. Some 30 participants from 13 countries presented 17 papers, all of which are included in these Proceedings. The first nine of them provide an overview of how multiple reservoir systems are operated in certain countries. The remaining papers report in greater detail on diverse case studies and provide discussion of some specific issues related to the subject of the workshop.

The presentations stimulated lively discussions on a whole range of topics, including the nature of interaction between system analysts and decision makers in a multiple reservoir system, institutional aspects of the decision-making process, methods and models that may be used for operation of multiple reservoir systems, and finally, assessment of further research needs in this field. A summary of the discussion and conclusions is presented at the end of these Proceedings.

The future work of IIASA, KGW-PAN, and IMGW will draw on the information exchange in Jodłowy Dwor. However, the value of the workshop extends beyond the work of the sponsors to that of the international water resources community at large. It is for this reason that these Proceedings have been assembled. It is hoped that this publication will find direct application in reservoir studies undertaken in various countries, and that it will stimulate additional research on the subject.

Z. Kacmarek
J. Kindler

CONTENTS

MULTIPLE-RESERVOIR OPERATION IN NORTH AMERICA

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INTRODUCTION

This paper focuses on the operation of multiple-purpose multiple-reservoir systems in North America. Our discussion of multiple-reservoir operation is divided into three parts. The first is a description of the principles and procedures currently used to operate multiple reservoirs in various river basins or regions. The second part describes the methods of analysis used and proposed for use for multiple-reservoir operation. The paper will conclude with some examples of a few existing multiple reservoir systems in Canada and the U.S., and their operating policies.

The operation of most multiple-reservoir systems in North America reflects the fact that there are sometimes conflicting and sometimes complementary multiple purposes served by the water stored in and released from reservoirs. These purposes can include:

a) Water Supply for municipal, industrial and agricultural (irrigation) needs from lakes and streams.

b) Water Quality Improvement by releasing water of higher quality upstream to dilute and transport downstream wastes.

c) Flood Control through the provision of available storage capacity during periods when floods are possible and maximum use of downstream channel capacities during periods of high runoff to reduce the likelihood of flood damage.

d) Hydropower Production by operating reservoirs so as to minimize loss of energy and meet energy and power requirements.

e) Navigation by insuring sufficient depth of water in navigation channels and sufficient water supply for lockages.

f) Recreation, whose benefits, while sometimes difficult to quantify in monetary terms, are nonetheless often present if appropriate pool levels and limits on level fluctuations are maintained.

g) Fish and Wildlife Enhancement through the maintenance of desirable pool levels or flows during critical periods in the year for greater fish and wildlife production and fishing and hunting benefits.

Assuming that it is possible to define ideal storage levels and downstream releases and/or diversions for every day, week or month throughout the year (i.e. assuming there exists a set of storage and release values that best satisfies all water users), reservoir operating procedures are needed and used to guide operators when it is not possible to satisfy these ideal conditions. Reservoir operating policies used in

North America usually include a definition of ideal conditions (with regard to storage levels, or releases, or both) and some guidelines for operation when these ideal conditions cannot be maintained, i.e. for non-ideal conditions.

Ideal storage volumes or levels in individual reservoirs are typically defined by "rule curves." When conditions are not ideal, operating policies or "rules of system operation" define what should be done for various combinations of system states and hydrologic conditions. Together, rule curves and rules of system operation define desired storage volumes or levels, reservoir releases, and diversion quantities. Ideal storage volumes or levels usually vary throughout the year, but do not vary from year to year. Similarly, releases or diversions are also expressed as functions of the time of year as well as the storage condition of upstream reservoirs. These functions or rule curves apply to reservoirs that are in a stationary state (in a probabilistic sense) and that are being operated under the same policy from one year to the next. The purpose of operating policies is to distribute any necessary deviations from ideal conditions in a manner that satisfies mandated laws or regulations and/or that minimizes the total perceived discomfort or hardship to all water users in the system.

There is a variety of operating policies in use at the present time. These operating policies vary from those that only define each reservoir's ideal pool level, or target level (and provide no information or guidance on what to do if maintaining those levels becomes impractical or impossible), to those that define very precisely how much water to withdraw or release at every control structure for all possible combinations of hydrologic and reservoir storage conditions. The next section will review the principal types of operating policies currently in use.

OPERATING POLICIES

Before reviewing various types of operating policies for the operation of multiple reservoirs designed to serve multiple purposes, some discussion of single purpose multiple-reservoir operation may be helpful. Consider the single-purpose of providing a reliable source of water. Throughout North America numerous such single-purpose reservoir systems exist. These systems are generally operated by municipal water supply agencies. For such systems various operating policies expressed in terms of release rates have been devised to minimize water wastage. These policies differ depending on whether the reservoirs are in parallel or in series, as illustrated by Figure 1 .

For single-purpose water supply reservoirs, the following simple operating rules have generally been adopted:

1. Reservoirs in Series - For such systems the downstream reservoirs are depleted before using upstream reservoir water to meet downstream demands. In Figure 1a , this would mean that the upstream reservoir (R_1) would not be drawn down to meet diversions D_2 and D_3 until the downstream reservoir (R_2) was empty. This procedure ensures maximum use of available storage and that no unnecessary lower reservoir spilling will occur.

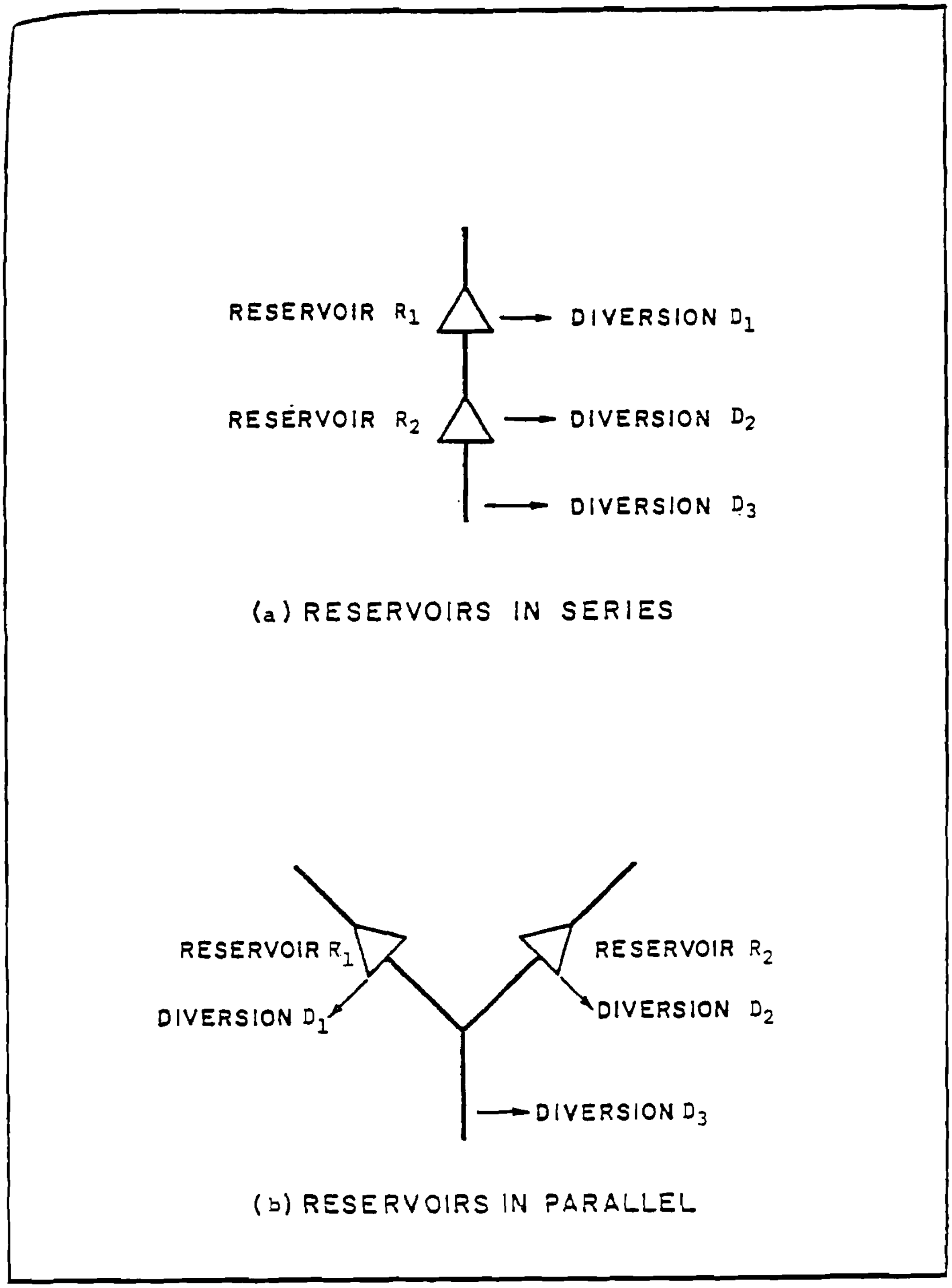


FIGURE 1. TYPES OF MULTI-RESERVOIR CONFIGURATIONS

2. Reservoirs in Parallel - Two procedures are commonly used. One involves discharging water first from reservoirs with relatively larger drainage areas (or potential inflows) per unit storage volume capacity. In Figure 1b, the drainage area to storage volume capacity ratios for the two parallel reservoirs are compared. The reservoir with the larger ratio is used to supply diversion D_3 before the other reservoir is drawn down. This procedure is valid only when the runoff per unit area is essentially the same in each reservoir's watershed. Discharging water first from the reservoir having the largest drainage area to storage volume capacity ratio will usually result in a reasonable conservation of water. Another, and more precise, procedure involves drawing in tandem from each reservoir in a manner that equalizes the probability of reservoir filling for each reservoir. This requires monitoring storage volumes and estimating future inflows. Such a policy minimizes expected water wastage.

For multiple-purpose reservoirs, or for single-purpose reservoirs involving recreation or hydropower, operating policies and associated rule curves commonly define the desired storage volumes and discharges at any time of the year as a function of existing storage volumes, the time of the year, demand for water or hydropower, and possibly the expected inflows. Such operating policies may include one or more of four general components.

1. Target Storage Levels or Volumes

These operating rules are limited to a prescription of the desired storage volumes or levels in each reservoir. Reservoir operators are expected to maintain these levels as closely as possible while generally trying to satisfy various water needs downstream. If the reservoir storage levels are above the target or desired levels, the release rates are increased. Conversely, if the levels are below target levels, the release rates are decreased. These release rates may or may not be specified but will depend in part on any maximum or minimum flow requirements and on the expected inflow.

Figure 2 illustrates a typical rule curve. The desired storage levels may be based on a compromise among recreational, fish and wildlife, flood control, hydropower and water supply interests. They are most often based on historical operating practice and experience sometimes supplemented by the results of simulation studies. Having only these target volumes or levels for each reservoir, the reservoir operator has considerable flexibility in day-to-day operation with respect to the appropriate trade-off among storage volumes and discharge deviations from ideal conditions, and on deciding from which reservoirs to withdraw water in order to meet downstream flow demands. Operating policies that are defined only by rule curves indicating ideal storage levels or volumes require experienced operators that have developed good judgment on how to minimize, over time and space, necessary storage volume and discharge deviations.

2. Multiple Zoning

Operation rules are often defined to include not only storage target levels, but also various storage allocation zones. For example, the following five zones might be considered:

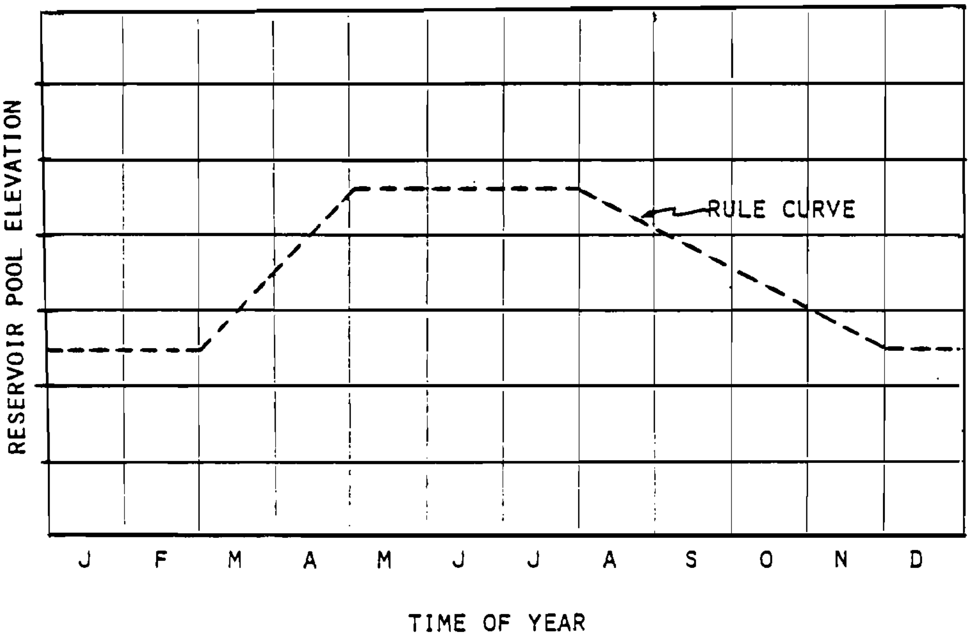


FIGURE 2. RULE CURVE DEFINING IDEAL STORAGE POOL LEVEL

(a) Conservation Zone - the zone of storage from which various water-based needs are satisfied. Water levels within this zone are generally satisfactory for recreational and environmental needs. The ideal storage volume or level is normally located within this zone.

(b) Flood Control Zone - a reserve for storing large inflows during periods of abnormally high runoff. When storage volumes are within this zone, downstream flows are increased temporarily to pass excess water out of the reservoir as quickly as possible.

(c) Spill or Surcharge Zone - the storage above the flood control zone associated with actual flood damage. Reservoir releases are usually at or near their maximum when the storage volume is within this zone.

(d) Buffer Zone - a reservoir beneath the conservation zone entered only in abnormally dry periods. When storage volumes are within this zone, downstream flows are decreased temporarily to satisfy essential needs only.

(e) Inactive Zone - the "dead" storage beneath the buffer zone which would, if possible, be entered only under extremely dry conditions. Reservoir withdrawals may or may not be possible, and if so, the withdrawals are an absolute minimum. Dead storage in excess of that below the sill of the water outlet structure may be required during some or all of the year to meet legal or institutional constraints.

Figures 3 and 4 illustrate such zones, which may vary throughout the year. The flood control zone is above curve B. If the storage level is in the flood control zone, the rule may provide for the maximum possible release if the storage level is above curve A, and the maximum release possible without causing flood damage when the storage level is between curve A and curve B. Reservoirs would be kept at or below curve B whenever possible for flood control purposes. Clearly if the need for flood control storage capacity varies throughout the year, the volume of flood control storage capacity should also vary, as is illustrated in Figure 4 .

Likewise, reservoir zones may dictate curtailing or reducing the allocation to lower priority uses when the storage volume falls below a specified level. Curve C of Figure 4 shows that storage level below which allocations to only critical or high priority uses would be maintained. Even further restrictions would be required if the storage level or volume were to fall below curve D in Figure 4.

Figure 5 illustrates the combination of zones and rule curve levels that may define the operating policy of each reservoir in a multiple-reservoir system.

These reservoir operating policies permit some flexibility in multiple-reservoir operation. To assist operators of multiple-reservoir systems, similar curves defining different release zones have been derived for groups of reservoirs. These multiple reservoir-system rules, together with the individual reservoir rules, offer additional guidance to those responsible for multiple-reservoir operation.

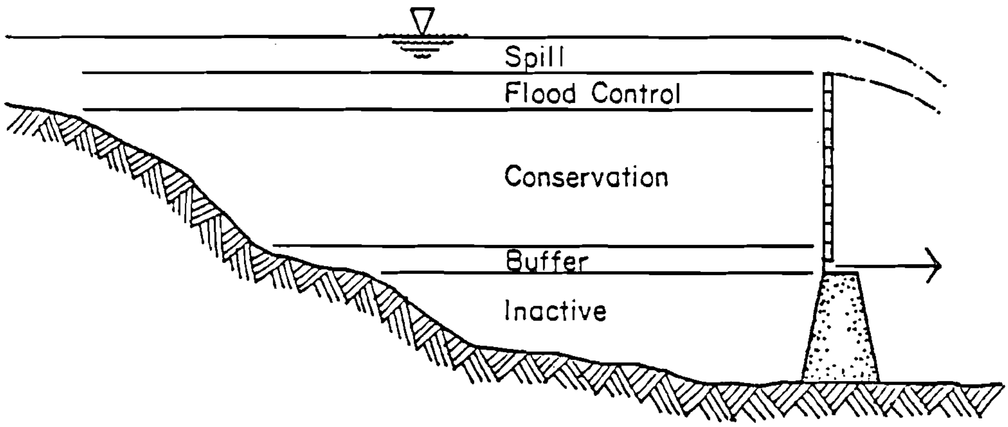


FIGURE 3. TYPES OF ZONES FOR INDIVIDUAL RESERVOIRS

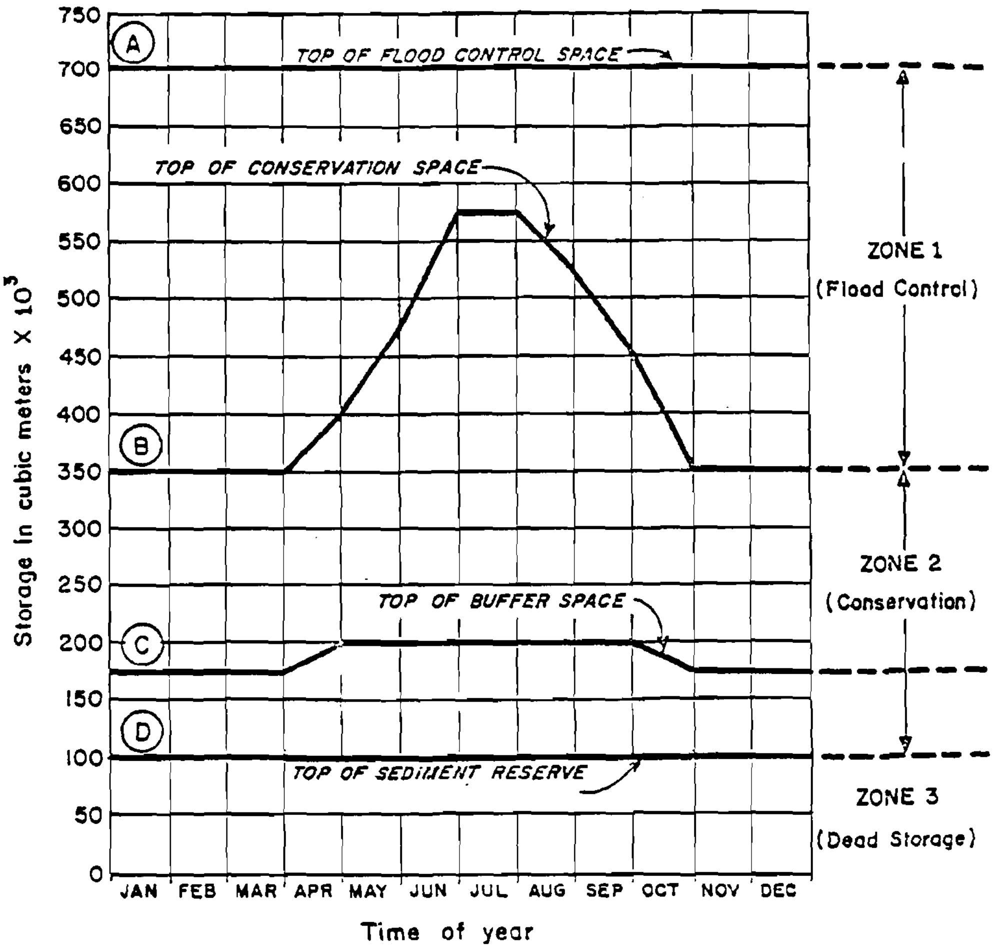


FIGURE 4. EXAMPLE OF SEASONALLY-VARYING STORAGE BOUNDARIES FOR A MULTIPURPOSE RESERVOIR

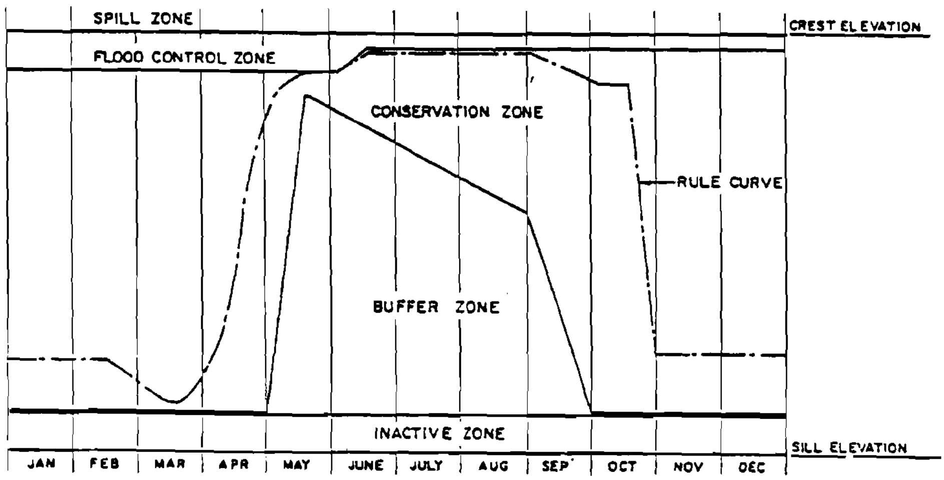
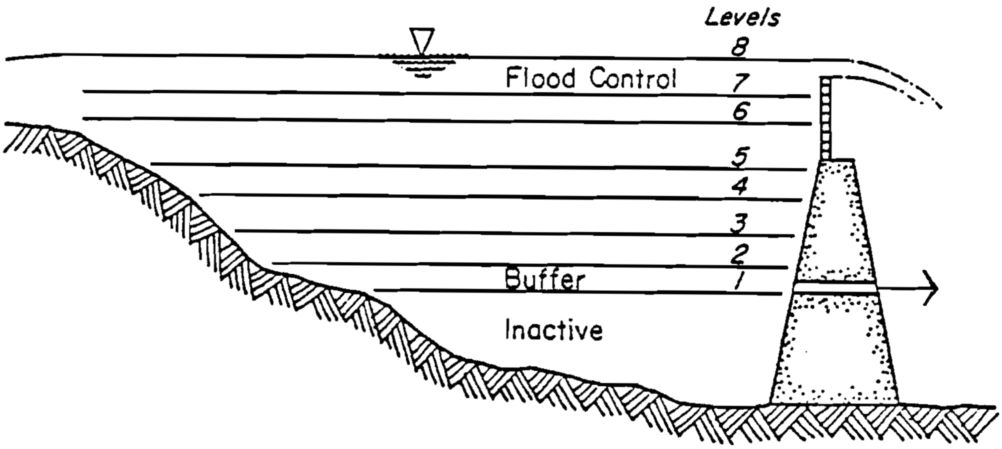


FIGURE 5. ZONES AND RULE CURVE FOR A TYPICAL RESERVOIR

A further aid in multiple-reservoir operation is provided by identifying multiple subzones within the conservation zone. Figure 6 illustrates such multiple subzones or levels. The volume within these levels can vary in magnitude, at a given time and over time. Their main purpose is for multi-reservoir storage-level balancing.

Using the zoning concept for reservoir operation, all reservoir storage volumes should be maintained in the same zone or subzone to the maximum extent possible. There are three basic concepts for such balancing of reservoir storage volumes. The first concept is based on keeping all reservoirs at their same zonal position, i.e. at a level where the percentage filling of the zone is equal for all reservoirs. This is sometimes referred to as the "equal function" policy. The second concept is based on a reservoir ranking or priority concept. The entire zone of the lowest ranking reservoir is utilized fully before starting on the next lowest ranking reservoir, and so on. The third concept is based on a "storage lag" policy. Withdrawals from the zones of some reservoirs are begun before withdrawals are begun from the same zones of other reservoirs. After a certain volume has been released from the initial group of reservoirs, releases are made from all reservoirs, maintaining the percentage difference of available zone volume. This policy is often used to provide a readily available reserve of water in case corrections in inter-reservoir balancing are needed after an unexpected or extreme hydrologic event.



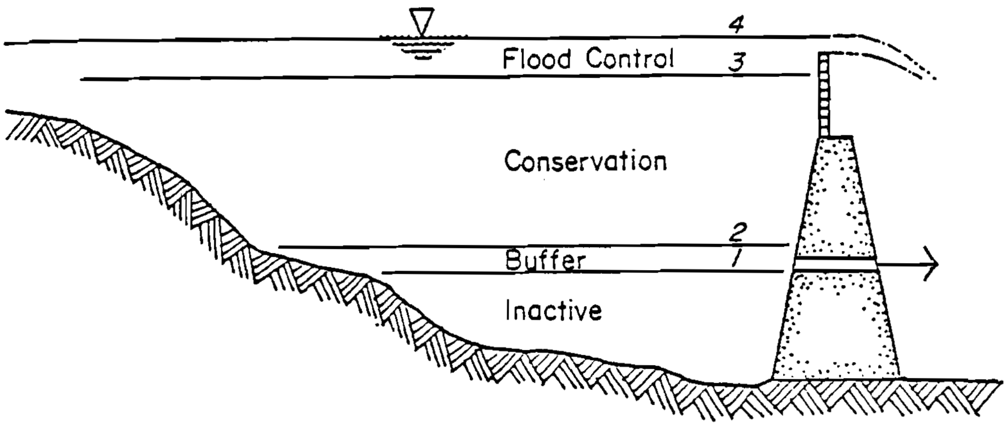


FIGURE 6. RESERVOIR STORAGE ZONES SHOWING CONSERVATION ZONE WITH AND WITHOUT MULTIPLE SUBZONES

Operating policies that are defined by storage zones and associated release rates and balancing procedures are much more prescriptive than policies defined only by reservoir rule curves. With only rule curves, the operators have substantial latitude and must use much more judgment in the operation of multiple-reservoir systems. Operational planning studies are oriented toward reducing this latitude by defining more prescriptive policies that will increase the likelihood that a system will be operated as optimally as possible.

3. Flow Ranging

This component of an operating policy provides a more prescriptive relationship between reservoir levels and channel flows. The reservoir release and/or diversion depends on which subzone or zone the storage volume is in. Instead of a possibly large reduction in the discharge from a reservoir when its storage volume falls from the conservation zone to the buffer zone, a sequence of smaller reductions can be specified, as the storage volume falls into progressively lower subzones or levels.

In addition, flow ranges for the individual channels downstream of the reservoirs can be defined as a function of upstream storage volume. As shown in Figure 7 , three such zones can be identified:

(a) Normal Flow Range - a flow range which can be considered ideal and that would be expected as long as all the upstream reservoir storage volumes were within their respective conservation zones.

(b) Extended Range - the enlarged range of flows that could be utilized if one or more upstream storage volumes were in either the flood control or buffer zones.

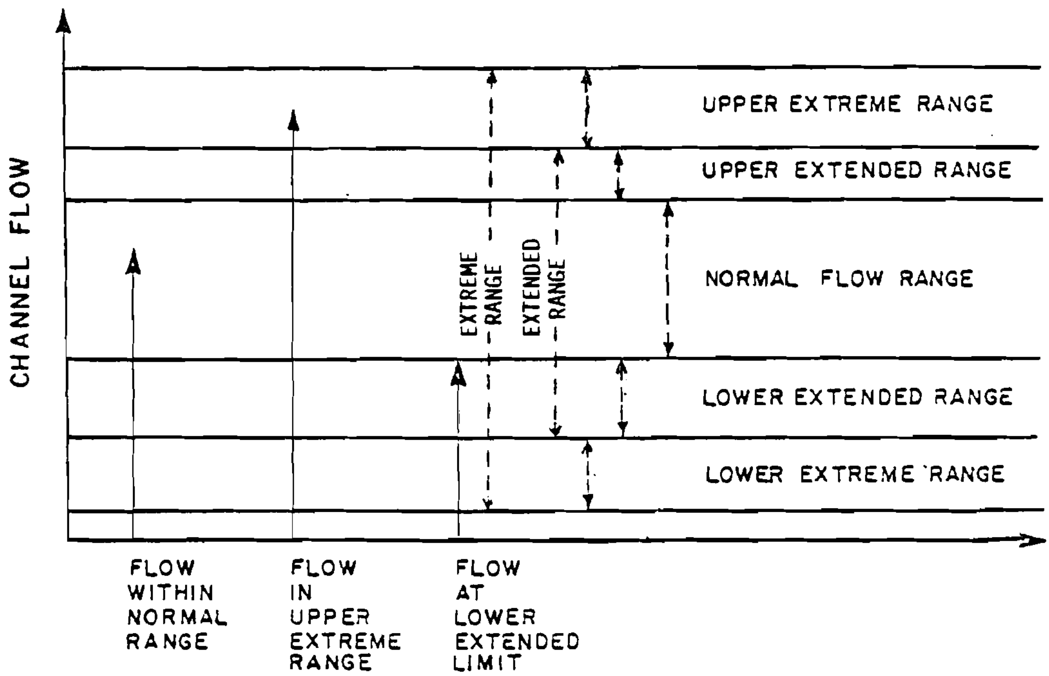


FIGURE 7. CHANNEL FLOW RANGES DEPENDING ON UPSTREAM STORAGE CONDITIONS

(c) Extreme Range - the further enlarged range of flow that could occur if one or more upstream storage volumes were in either the spill or inactive zones.

These flow ranges can be time-dependent, as can be the reservoir sub-zones.

Given both multiple zoning for storage volumes and flow ranging for channel flows, there is less need for operator judgment when balancing reservoir levels with channel flows and keeping within the restrictions imposed by these zones or levels and flow ranges.

4. Conditional Rule Curves

In some cases conditional rules have been defined for multiple-reservoir systems. These policies define reservoir releases not only as a function of the existing storage volumes and time of year, but also as a function of the expected natural inflows into the reservoirs for some prespecified time period in the future. Such policies can be described as functions, in tabular form, or as a diagram. Figure 8 illustrates some conditional flood storage capacity zones, depending on the winter snow depth or on the recent precipitation record prior to a possible flood period. While approximate methods for determining these conditional rules exist [7], research continues towards finding improved methods for defining conditional operating policies for multiple-reservoir systems.

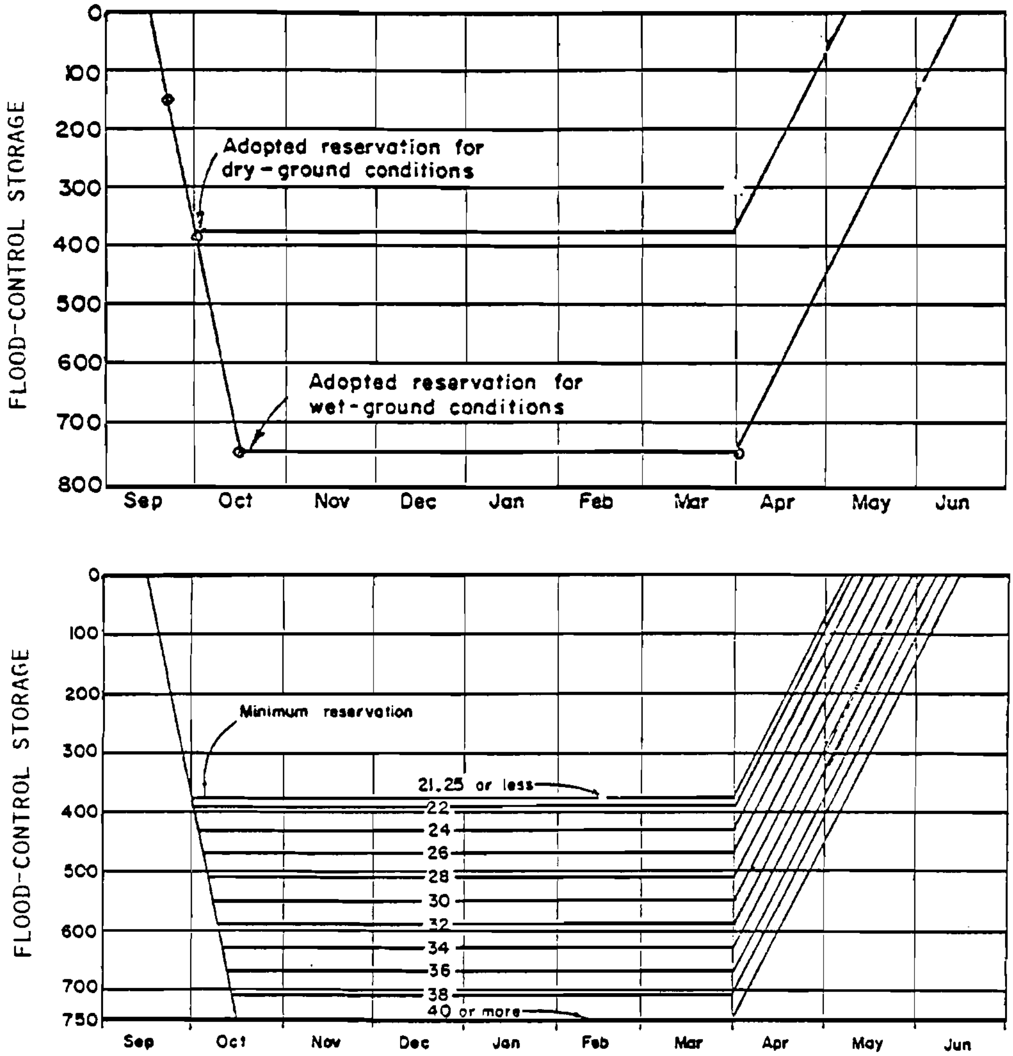


FIGURE 8. CONDITIONAL FLOOD CONTROL STORAGE ZONES BASED ON GROUND CONDITIONS, PREVIOUS PRECIPITATION, SNOW DEPTH, ETC.

In addition to the four general components of operating policies and their modifications as discussed above, there is also the use of computer programs developed to be run each time a new release decision is to be made, as an aid to those responsible for multiple-reservoir operation [63,72]. Input data for these programs usually include flow forecasts, the current state of the reservoir system, the system operating policies, and appropriate objective functions for reservoir operation. The program output includes computed releases at each reservoir site or control structure that will best satisfy the prescribed operating objectives. When revised estimates of future inflows, storage volumes, and possibly economic environmental or ecological parameters are obtained, the program is rerun to obtain new estimates of appropriate reservoir releases, and their respective impacts. This process can be repeated at regular intervals (daily or weekly or even hourly during flood events).

OPERATING POLICY ANALYSES

Over the past several decades, increasing attention has been given to the use of mathematical (simulation and optimization) models for deriving operating policies of multi-reservoir systems. In some cases, with only small improvements in system operation (for example, only 1 or 2 percent increase in hydropower production), millions of dollars of additional annual economic benefits can be realized. This appreciation has been coupled with a substantial research effort through the years, and has led to continuing developments in the conceptual thinking and the mathematical formulations for a variety of models. As a result, there are now available a vareity of methods for analyzing the operation of multi-reservoir systems used to satisfy collective water-based needs of river basins.

The development of mathematical models for deriving optimal policies for scheduling releases for multi-reservoir systems has been much more difficult compared to that for single-reservoir systems. Much of the early developmental work was directed at translating the release from a single reservoir into equivalent economic benefits. Optimization or simulation models were then used to develop time-based patterns of releases so that the total of the benefits over time was maximized. Many of these early developments were performed with either linear programming (LP) or dynamic programming (DP) optimization procedures.

These early single-reservoir operating models, however, proved to be both time consuming and expensive. In some cases, several hours of computer time were required to obtain an optimal solution, even when analyzing only a single reservoir. In analyzing two or more interconnected reservoirs, the problem, while easily modeled, often proved to be virtually insoluble from a computational viewpoint. It is still not possible to obtain an explicit multiple-reservoir operating policy that specifies the release that should be made from each reservoir as a function of a) the current storage volumes in all reservoirs, b) the time period, and c) the actual or expected natural inflows when these inflows are uncertain. Recent developments, however, have indicated considerable promise in using optimization models for developing rule curves for systems with several reservoirs, or for indicating the releases to be made from each reservoir on a real-time basis.

When considering more than two or three reservoirs, it has been necessary to adopt a different modeling strategy than that used for single-reservoir systems. Most of the work to date has focussed on the use of simulation models, but limited use has also been made of optimization models for estimating policies which can then be more accurately evaluated using simulation. Since simulation models do not define the optimum policy or procedure to be used directly, it is necessary to use a trial-and-error procedure to search for an optimal or near optimal solution. To acheive this, it may be necessary to perform a large number of simulation runs -- which can of course be computationally expensive.

Simulation models, however, have certain other advantages. They usually permit more detailed representation of different parts of the system (such as detailed responses of individual reservoirs and channels or the effects of certain time-varying phenomena). They also allow added flexibility in deriving responses which cannot always be readily defined in economic terms (recreational benefits, preservation of fish and wildlife, etc.). Finally, they provide an effective focus for dialogue with system operators (the ideas inherent in simulation modeling can usually be understood more easily than the ideas in optimization modeling).

To provide a brief state-of-the-art overview of various modeling strategies which are being used to define policies and procedures for scheduling releases from multi-reservoir systems, the models have been separated into three general groups:

- optimization models for single reservoirs;
- optimization models for multi-reservoir systems;
- simulation models.

These will be discussed in turn.

Single-Reservoir Optimization Models

The early concepts for defining reservoir releases were based on adaptations of inventory theory. The initial connection was developed by Little [37] in 1955, who used a DP approach to develop an operating policy for minimizing power production costs in a mixed hydroelectric-thermal system. Manne [43] showed that LP could also be adapted to inventory problems. Later, he showed how this method could be used for deriving reservoir release policies in which the supply is uncertain. He represented time as a series of individual time intervals and then considered the release in each period to be a function of storage at the start of the period and of average inflow rate during the period [44]. In 1961, Thomas and Watermeyer [69] used a slightly different approach, but again used LP to solve the same problem. They assumed that inflows had known probability distributions, but were independent or serially-correlated random events [71]. Others adopted the Thomas and Watermeyer approach in principle and carried out more detailed investigations [14,25,38].

In parallel with developing the use of LP models for defining optimal release policies, other techniques were being pursued. In 1962, Bather [4] developed an approach based on the use of DP. Falkson [11] also developed an approach which is based on the combined use of LP and DP and is referred to as the "policy iteration" approach. In 1963, Buras [10] used DP for scheduling releases from a combined reservoir-aquifer system.

All the models described above can be classified as being "explicit stochastic models," i.e., they use probability distributions of inflow directly in deriving optimal release policies [54].

Despite the various techniques which were developed, many of the models proved to be very expensive from a computational viewpoint. In their 1970 paper, for example, Gablinger and Loucks [25] showed that a single reservoir operating problem in the northeastern U.S., if solved using LP, required approximately 2,000 equations, 15,000 variables, and 2 hours of computer time (on a 360/65 computer). Although the same solution would be obtained more efficiently with the use of DP, such a model would require more programming effort. Loucks and Falkson [41] compared the use of stochastic LP, DP, and policy iteration methods. They concluded that the use of LP to determine sequential operating policies for large multi-period problems was the most expensive computationally and that, for all practical purposes, its use was limited to analyzing only single-reservoir systems in which the number of possible discrete storage volumes, inflows and time intervals was relatively small. Although the other two methods were also computationally expensive, they appeared to show more promise in applications to multi-reservoir systems.

During the 1960's, there were also developments in "implicit stochastic models," i.e., models which optimize returns for stochastic hydrologic sequences, but which assume that these sequences are known a priori. Some early developments in this area were conducted by Hall [27] and Hall and Buras [28]. Their models were solved using DP methods. In 1966, Young [73,74] extended the results of these earlier investigations. His approach included streamflow synthesis, deterministic optimization (again with the use of DP) and regression analyses. The regression analyses were used to define release values in terms of storage levels and previous inflow rates. The data used for the regression analyses were derived from the sequence of computed responses obtained from the optimization model.

Although Young's work was directed at analyzing only a single reservoir, it was considered that the "implicit stochastic" approach would be superior to the "explicit stochastic" approach for multi-reservoir systems. For the implicit approach, the computational effort in optimization is directly proportional to the number of reservoirs in the system. Computing time grows exponentially with the explicit approach.

There are, however, certain theoretical questions which still remain unanswered in using the implicit approach. For example, the form of the equation (what independent variables should be included and how they should be treated) for regression analysis is continually open to question. To date, there has not been any attempt to assess the error resulting from using an optimal operating rule derived by this process compared to using the theoretically optimum operating rule. Furthermore, it may never be possible to derive the theoretical optimum, since even the explicit approach introduces error in discretizing the probability distribution. For all practical purposes, however, it is unlikely that the use of the implicit approach would produce significant errors if used to derive optimal reservoir operating policies.

Multi-Reservoir Optimization Models

Since the early development of single-reservoir optimization models, considerable work has been carried out in extending some of the modeling strategies to multi-reservoir systems. As envisaged, the amount of development based on using the explicit stochastic approach has been limited. In 1968, Roefs [53] demonstrated that this strategy led to increasing computational effort as the number of reservoirs increased. One known application using this method on a multi-reservoir system was performed by Schweig and Cole [56]. They applied DP to a two-reservoir system and found that computational costs were high, even when using only very simplified streamflow representations. Similar results were found by Gablinger [24] and Houck and Cohon [32].

Various general approaches for multi-reservoir systems have been used with the implicit stochastic approach. In one of the earliest developments, Hall and Roefs [29] optimized the operation of the three-reservoir Oroville-Thermalito power generating complex in northern California. This optimization was performed with DP on a specific 6-year hydrologic sequence. Although the approach was successful, it proved to be computationally expensive.

In a related study, Parikh [50] explored the use of a strategy which he referred to as "linear dynamic decomposition programming." In this approach, he combined the use of DP for optimizing individual reservoirs and LP for combining the reservoirs collectively into an integrated optimization model. The approach uses dual variables from the LP solution to constrain the DP solution. In turn, the DP solution provides column vectors for the LP matrix. An optimal solution is obtained after a series of iterations back and forth between the LP and DP portions of the model.

Parikh used his model for analyzing two test problems: a two-reservoir system for 24 months of hydrology, and a four-reservoir system for 36 months of hydrology. For both problems, the solution came close to being optimal relatively quickly. However, a substantial number of iterations was conducted before finally reaching the optimal solution. Although the computational effort was substantial, it was not prohibitive. This method has therefore demonstrated some potential for application to larger systems.

In parallel with this research effort, Buras [11] developed a simplified version of the Sacramento Valley multi-reservoir system in northern California for employing the Parikh model. His model included a four-reservoir system and 10 years of hydrologic input.

After this introductory work, several modifications to the modeling strategy were carried out. Hall, et al. [30] explored the potential for making the Parikh model more efficient. Instead of using dual variables for the DP portion of the model, it was suggested that better efficiency could be achieved by defining mathematical constraints. This idea arose from the knowledge that computational time for DP models reduces as the problem becomes more constrained, up to a point.

In another modification of the Parikh model, Roefs and Bodin [54] introduced additional considerations in an attempt to obtain a more realistic representation of problems in practice. In particular, these included the effects of changes in hydro-electric energy production rates for representative reservoir drawdown conditions and nonlinear release-benefit relationships. While Roefs and Bodin achieved some success with their strategy, they concluded that the approach was computationally expensive. For example, one particular model run required approximately 20 hours of 360/50 computer time before being terminated!

During the late 1960's it became apparent that many of the strategies being examined were both too expensive and too impractical for most problems in practice. Simplification was clearly required. In 1969, an interesting idea -- referred to as the "linear decision" rule -- was proposed by ReVelle, Joeres and Kirby [51]. This rule had been proposed earlier by Charnes, Cooper and Symonds [12] for determining refinery rates for heating oils to meet stochastic weather-dependent demands. For a reservoir system, ReVelle, Joeres and Kirby suggested that the reservoir release, r_t , during a particular time period, t , could be related to the storage, s_t , at the start of the time period by the linear relationship

$$r_t = s_t - b_t$$

where b_t is a decision variable to be derived by the model. This rule had the decided advantage that it could be translated conveniently and efficiently into LP formulation.

Since its introduction, however, the linear decision rule has been a subject of considerable controversy. Revelle and Kirby [52] , Joeres, Liebman and ReVelle [33], Nayak and Arora [46,47], Eastman and ReVelle [16] and Leclerc and Marks [36] have modified, extended and/or applied this method to reservoir management problems. However, Eisel [17], Loucks [39], Sobel [64] and Loucks and Dorfman [42] have all questioned the utility of this model for reservoir management. For example, Loucks and Dorfman [42] have demonstrated that the use of the decision rule generally produces conservative results, primarily because the imposition of the rule itself represents an additional operating constraint in the system. This conclusion applies even to the less conservative linear decision rule that includes the current inflow on the right-hand side of the above equation. They suggest that while this technique may be suitable for screening studies, it is not in itself satisfactory for deriving optimal operating policies for single or multiple reservoirs.

One further development is also worthy of note. In parallel with developing the Texas Water Plan in the late 1960's and early 1970's, a three-phase research program was implemented for developing a computer-oriented planning methodology for use in the planning of large multi-basin systems. This methodology was developed using the Texas Water System as an example.

One of the many models developed in this study is an optimization model (referred to as the Allocation Model) which uses the "out-of-kilter" algorithm [19,67]. This algorithm, which was developed by Fulkerson [15, 21, 23], is used to solve a special class of LP problems, each of which can be represented as a "capacitated network," i.e., as a series of nodes and interconnecting arcs. The objective is expressed as the minimum collective cost of flows through all arcs, subject to two types of constraints. The first type is simply the equation of continuity at each and every node, i.e., the sum of flows into each node must equal the sum of flows out of the node. And the second set states that every arc flow must be between some prescribed lower and upper limits. Fortunately, many water resource problems can be transposed directly into an equivalent network representation. Storage changes in reservoirs during individual time periods and changes in system operation through a sequence of time periods can also be represented effectively. In the Texas Water Study, the out-of-kilter algorithm was used for defining optimal operating policies for an 18-reservoir system with 42 links for a 36-year hydrologic period. Although this approach was still computationally expensive, it was estimated that the out-of-kilter algorithm was about 20 times faster and required 35 times less computer storage than a standard LP algorithm.

Within the last several years a modeling approach, using LP, for defining "firm" yields throughout a river basin has been developed and applied to several planning problems [40]. This approach lends itself to the estimation of the storage zones, and their associated release restrictions, for each reservoir. These estimates of storage zones can later be adjusted based on more accurate simulation studies.

The so-called yield model uses two sets of storage continuity equations for each reservoir. One set determines the overyear storage requirements, if any, based on annual flows and one or more yields, having prespecified reliabilities, to be derived from the reservoir operation. The other set defines the within-year storage requirements, if any, that are determined from the within-year inflow and yield distributions in a critical year. Each yield defines a separate storage zone at each reservoir. The total volume in each zone at the beginning of each within-year period is the sum of the required overyear and initial within-year volumes derived from the yield model.

Figure 9 illustrates the use of this modeling approach for defining operating rules for a three-reservoir system. On an interactive computer graphics terminal, the three reservoir system has been "drawn in" (Figure 9a), and the inflows and two required yields and their annual reliabilities are defined. Figure 9b illustrates the display of a portion of the model solution, on the graphics terminal, in the form of operating zones for one of the three reservoirs. The results of operating this reservoir, along with the others, using these storage zones and the "historical" flows, are shown in Figure 9c. The model provides a first estimate of a multi-reservoir operating policy in the form of storage zones, including that for flood control, if applicable. Using interactive graphics the derived operating policy can easily be modified and resimulated numerous times for possible improvement.

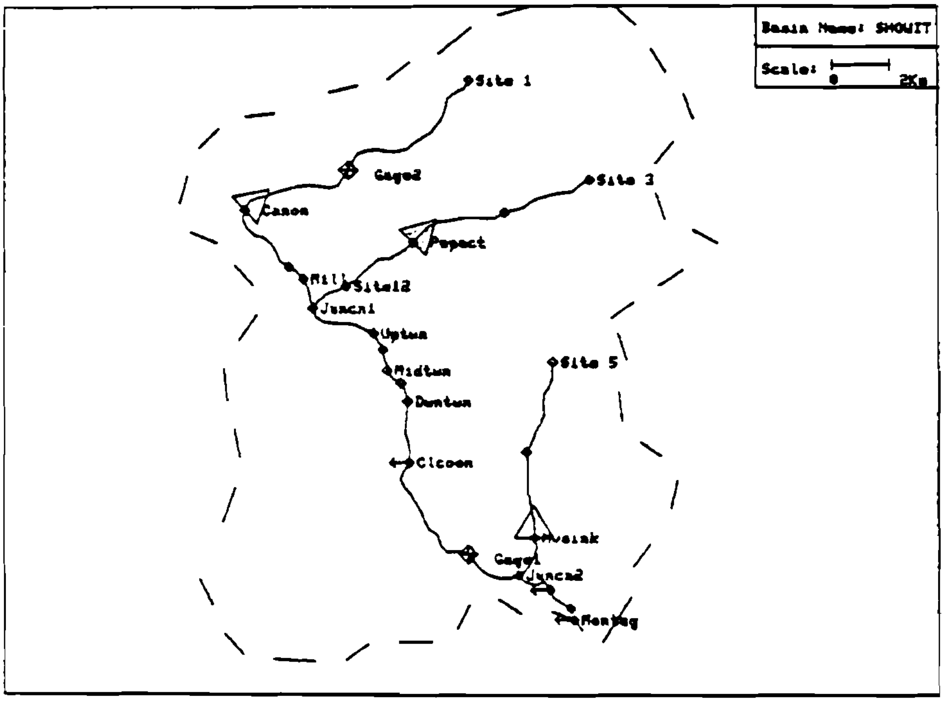


FIGURE 9a. A THREE-RESERVOIR OPERATING PROBLEM FOR WHICH RESERVOIR STORAGE ZONES ARE TO BE DEFINED AND THEN SIMULATED.

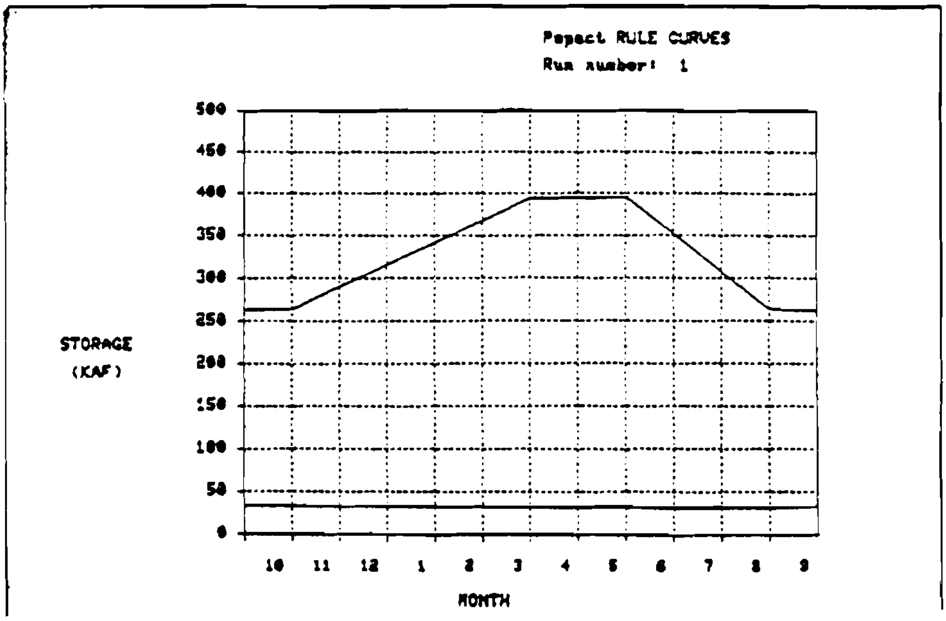


FIGURE 9b. RESERVOIR STORAGE ZONES FOR TWO YIELDS WITH DIFFERING RELIABILITIES DERIVED FROM AN OPTIMIZATION MODEL FOR ONE OF THE THREE RESERVOIRS IN FIGURE 9a (Pepact).

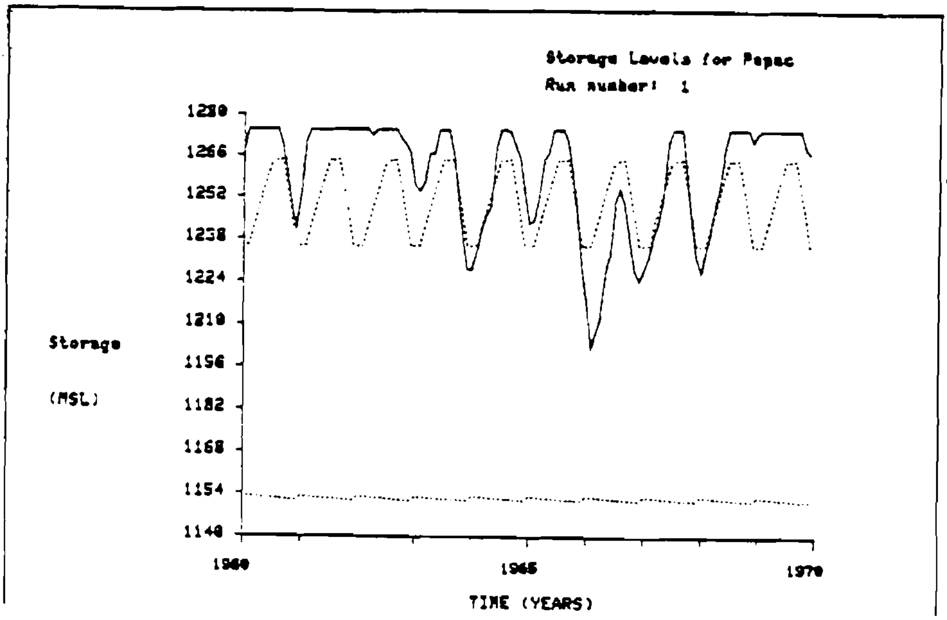


FIGURE 9c. SIMULATION OF PEPACT RESERVOIR STORAGE VOLUMES OVER CRITICAL YEARS OF HYDROLOGIC RECORD. (Derived Storage Zones Shown in Dotted Lines)

Multi-Reservoir Simulation Models

Simulation models continue to be used extensively for analyzing water resources systems. This is especially true for systems with many reservoirs as well as for those which have nonquantifiable benefits. While there are literally thousands of simulation models being used in practice, five recent models are of special interest. The first of these is the HEC-3 model developed by the U.S. Corps of Engineers [5,70]. The purpose of this model is to simulate the response of water resource systems designed to simultaneously satisfy a variety of water-based needs. This model is sufficiently flexible to include any arbitrary configuration of reservoirs and channels. The algorithm searches through the system in the upstream to downstream direction, determining each system requirement in turn and the amount of that requirement to be satisfied by each reservoir. Since individual project responses are not known until the entire system is searched, it is usually necessary to make three sequential searches through the entire system in each time interval in order to achieve the desired reservoir balancing. The model then proceeds to the next time interval (monthly time intervals are typical) and the process is repeated. After proceeding through all time intervals, which may include several years of hydrology, simulated responses are appropriately summarized.

One particular development in HEC-3 is of special interest. While the idea of maintaining time-based rule curves to denote ideal operating levels for each of the various reservoirs was retained, this was supplemented with the idea of reservoir zoning (see Figure 6a). Each reservoir would have a number of zones (typically about 6), with each zone representing a specific level range. The algorithm was then structured so as to bring all the reservoirs to the same zonal position if the optimal (or rule curve) level could not be attained. This idea permitted considerable flexibility in representing a variety of different operating policies. These included both reservoir ranking as well as policies based on ensuring that deviations from optimal operating levels were distributed in some equitable manner.

The HEC-3 model has been used extensively in practice. This is due not only to both the general and flexible nature of the HEC-3 program, but also to the fact that the model is well documented and well supported. Representative applications include the Corps studies of the Willamette River system in Oregon and the series of operational studies on the Arkansas-White-Red system in the southern United States [6, 22]. For the Arkansas-White-Red system, one of the more recent representations consisted of 18 reservoirs, 15 service locations and 8 hydroelectric power plants. Water-based needs included hydropower, navigation, recreation and flood control. The model was used to derive optimal operating policies by simulating various strategies for a 21-year hydrologic sequence. Further detail will be presented later.

A second model which is also of special interest is the SIMYLD-II model which was developed in the research portion of the Texas Water Study [68]. This model is a multi-reservoir simulation model. In each time interval, however, an optimization submodel, using the out-of-kilter algorithm, is used to define the optimal operating strategy. The objective of the submodel is to minimize system costs (primarily pumping costs) in each time interval. Policies of operation are represented by varying the limit constraints of each arc -- which denote either reservoir releases or storage values.

A third model is the multi-reservoir model developed for the Oswego system by the New York State Department of Environmental Conservation [65]. This particular model is of interest because it extended some of the basic ideas of multi-reservoir zoning inherent in the U.S. Corps HEC-3 model. The number of zones was reduced to four. These were referred to as the flood control, conservation, buffer and inactive zones (see Figure 6b). The flood control zone was used as temporary storage for alleviating downstream flood damage during periods of excessive inflow. Similarly, during periods of abnormally low inflow, the buffer zone could be used for releasing minimal flows to satisfy essential downstream needs only. The conservation zone represented the zone of normal operation, with the ideal operating level being implicitly positioned at the top of this zone. The inactive zone, positioned under the buffer zone, defined the range of levels which are usually not available for regulation purposes. The algorithm for the Oswego simulation model was based on maintaining all reservoirs at the same zonal position, if ideal operating levels could not be achieved (similar in concept to the HEC-3 model). Downstream flows were adjusted in accordance with the zonal position of the upstream reservoirs. However, since the model was designed specifically for the Oswego system, it cannot readily be adapted to other multi-reservoir systems.

The fourth model is the Acres multi-reservoir model, which was initially developed for exploring alternative strategies for operating the Trent River Basin in Ontario, Canada [1,2,58,62,63]. The algorithm for this model was an adaptation and extension of the basic ideas contained in all three models discussed above. It included the combined rule curve-zoning representation which was inherent in both the HEC-3 and

Oswego models. However, this representation was extended by including an additional "spill zone" and by having the rule curve positioned anywhere in the conservation zone (and not necessarily only at the top of this zone). Additional flexibility was achieved by representing flows in the various channels by a series of flow ranges. This permitted not only a balancing of the relative levels in the individual reservoirs, according to equal function, priority ranking or storage lag policies, but also a general balancing of reservoir levels with channel flows.

As with the SIMYLD-II model, the Acres model used the out-of-kilter optimization routine as a submodel for achieving optimal responses during individual time intervals. However, instead of minimizing system cost, which the SIMYLD model did, the objective function in the Acres model was designed to reflect the chief operator's optimal decision and monitoring process for a particular operating policy. For any given hydrologic condition, it was perceived that the operator would minimize a collective sum of penalized deviations from ideal operating conditions for the system as a whole. Each of the deviations, which were either violations from reservoir rule curves or channel flows outside "normal ranges," was penalized with representative "penalty coefficients." By assigning appropriate values to the various penalty coefficients, it was then possible to reproduce the system response which the operator would achieve for the prescribed operating policy and given hydrologic conditions.

The Acres model, which was structured for any arbitrary configuration of reservoirs and interconnecting channels, has been used as an aid in defining reservoir operating policies for eight separate river basins. It has also been modified slightly and is now being used as a day-to-day operating tool for defining reservoir releases in the Trent River System in Ontario[63].

The fifth, and perhaps the most commonly used of all reservoir simulation models in North America, is the HEC-5 computer program titled Simulation of Flood Control and Conservation Systems. This program, like HEC-3, was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center at Davis, California [16a]. As the title suggests, the model simulates the operation of any spatial configuration of multiple reservoirs within a river basin, and can be used for studying proposed operating policies for both conservation and flood control purposes.

HEC-5 operation for flood control is based on the release of waters from the seasonal flood storage capacity of each gated reservoir as quickly as possible without exceeding certain predefined maximum flows that would cause flood damage at various downstream sites. Where the choice of which discharge rates should be made from which reservoirs, the decision is based on a prespecified balancing rule, similar to those used to balance conservation storage volumes in multiple reservoirs. Streamflow routing effects are considered, as they together with the discharge rates determine the spatial and temporal distributions of flows downstream from various reservoirs.

The HEC-5 conservation operation attempts to meet all downstream demands without wasting water. The program time steps can be as short as 1 hour for flood control, or as long as one month for conservation operation. During flood periods these time sequences can be combined to consider flood and conservation operation simultaneously.

While the model is primarily for hydrologic simulation it can also be used to evaluate economic effects of flood control and hydropower. Through simulation of alternative operating policies, rule curves can be improved and the sizing and location of potential reservoirs can be studied. HEC-5 provides a means of accurately simulating and refining the results of any optimization model developed and used for the preliminary definition of multiple reservoir operation policies. The model is well documented and maintained for anyone's use by HEC. During 1979 over 500 executions of HEC-5 were recorded per month on the HEC-maintained HEC-5 program and over 70 source decks were distributed.

Multiple reservoir simulation models used to assess the impact of various operating policies are useful only if the multitude of data derived from all simulations can be compared and evaluated. Obviously the means and variances, and even the time distribution, of numerous site-specific variables such as reservoir storage volumes and releases, and their associated benefits or losses, can be computed and used for policy evaluation. Methods have also been proposed to permit an evaluation based on system reliability, resilience and vulnerability [30a]. Reliability is a measure of how often a failure, however defined, occurs. Resilience is a measure of how quickly the system recovers from failure; and vulnerability is a measure of the magnitude or consequences of failure, should failure occur.

Reliability is a widely applied concept in water resources planning. Resilience and vulnerability are relatively new criteria. If a system failure tends to persist once it has occurred, it may have serious implications even though such failures occur only infrequently and hence the reliability is high. The associated operating policy may be less desirable than a policy which results in a lower reliability but a higher resilience, as illustrated in Figure 10.

Both resilience and vulnerability, the likely magnitude of failure should it occur, can be expressed in a variety of ways. Since they are random variables it is possible to define their expected values or their values that are exceeded with a specified probability. Figure 11b illustrates the relationship among reservoir release reliability, resilience and vulnerability associated with changes in two parameters of an operating policy defined in Figure 11a.

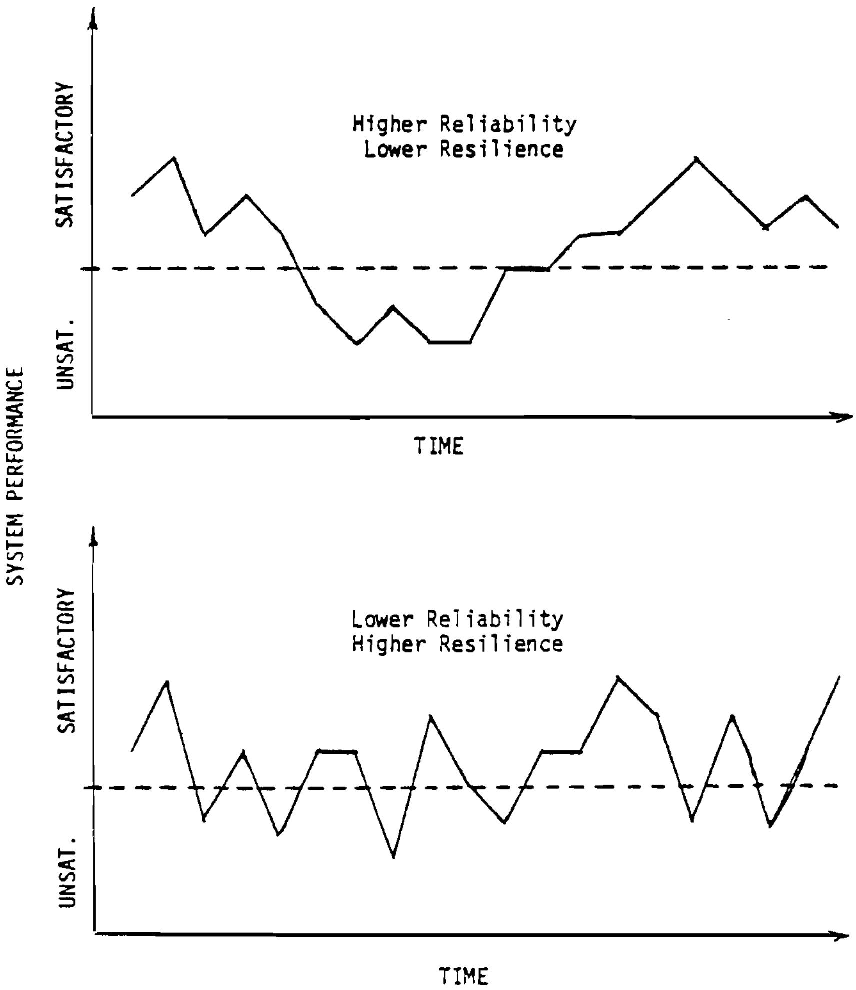


FIGURE 10. ALTERNATIVE TIME SERIES OF PERFORMANCE VALUES SHOWING CONTRASTING SYSTEM RELIABILITY AND RESILIENCE

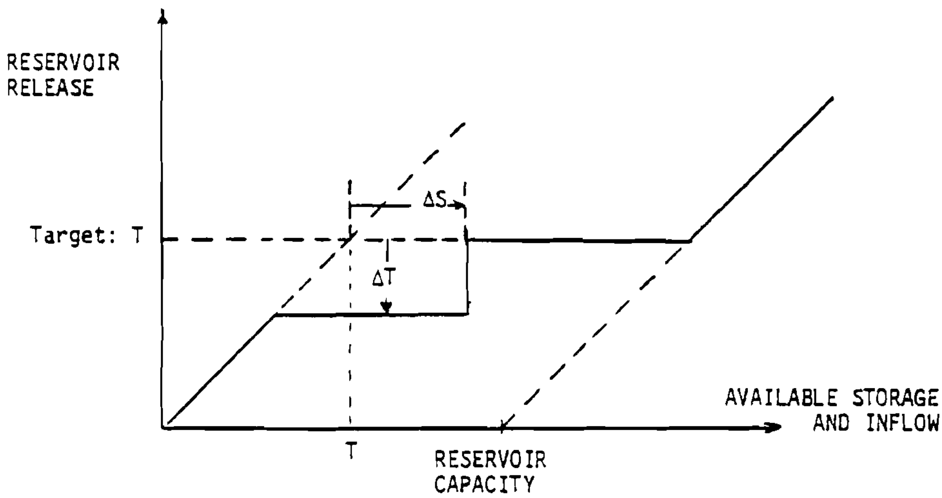


FIGURE 11a. RESERVOIR OPERATING POLICY HAVING PARAMETERS ΔS AND ΔT

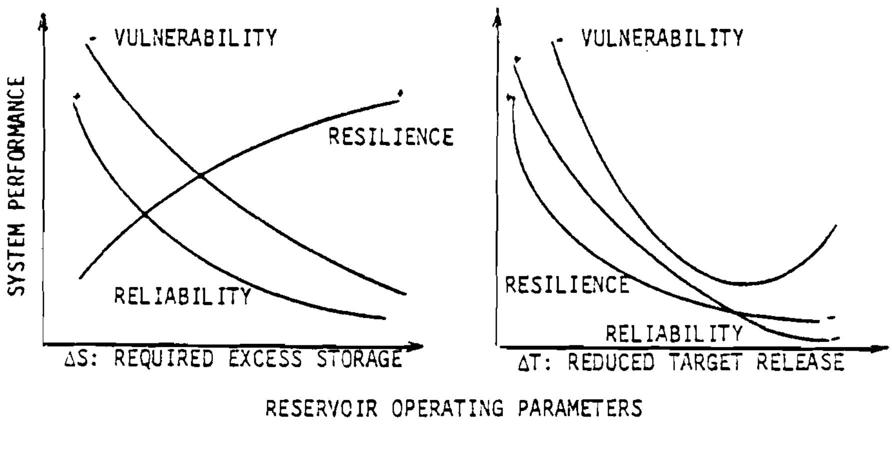


FIGURE 11b. RELATIVE PERFORMANCE CRITERIA VALUES AS FUNCTIONS OF OPERATING POLICY PARAMETERS ΔS and ΔT

One final comment on the use of simulation modeling is appropriate. In many systems, the perception of "what is an improvement in system operation" is still subject to collective perception and interpretation. This is especially true for systems where the principal benefits are considered to be nonquantifiable (examples include recreation, fish and wildlife preservation, low-flow augmentation, etc.).* For developing improved operating procedures in such systems, it is desirable (if not mandatory) therefore to include the interpretation of system operating staff.

In some of its recent studies, Acres has given special attention to this aspect. Acres' approach has consisted essentially of two sets of simulations. For the first set, the response of the system (in terms of reservoir levels and releases) as reflected in recent operating practice has been simulated. In essence, this is the model calibration phase. These results are obtained by an iterative process of system simulation with the model, and by extensive and detailed discussions with operating staff. After calibrating the model, a second set of simulations is then obtained, including system responses for various alterations in operating policy. These runs are based on the collective interpretation and judgment of system operating staff and systems analysts as to what constitutes improvements in system operation. This strategy has proven to be very successful in practice. It also has the decided advantage that at the time of implementing changes in operating procedure, the operating staff are more likely to support and implement the changes (since they fully understand and appreciate the total implications of these changes).

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* This is not to imply that these benefits cannot be quantified. Rather it is meant to include the perceptions of some management groups who find it more comfortable to consider certain benefits in qualitative rather than in quantitative terms.

Future Research

It is apparent that we still have not solved the general multiple-reservoir operating problem. There are substantial difficulties in identifying policies that are both truly optimal and computationally tractable. Given the substantial economic gains that can be realized with only a very modest improvement in operating procedure, there are strong economic incentives for continuing research in this field. The substantial ongoing investments by the Tennessee Valley Authority [35], the Central Valley Project [45,72] and the Columbia River System [26] to improve reservoir operating policies with the use of better mathematical models are indications of the confidence being placed in recent modeling developments.

While the explicit stochastic optimization approach appears to be the only technique available at this time for producing truly optimal solutions (aside from errors in defining the objective or in discretizing the probability distribution), it is for all practical purposes computationally intractable for anything except a single reservoir. Because of the rate of increase in computational effort with the increase in the number of reservoirs, this technique shows little promise for future application to real-life systems.

On the other hand, the implicit stochastic optimization approach shows greater promise. The development and use of implicit stochastic optimization models is still being confined to small multi-reservoir systems. In the TVA study, for example, the assessment of alternative optimization strategies has focused on only a 6-reservoir system representation. For such systems, containing a reasonably small number of reservoirs, the answers derived could lead to substantial improvement in system operation over present practice.

Improvements continue to take place in the development of implicit stochastic optimization models. The algorithms undergoing continuing development include the out-of-kilter algorithm, various modifications of DP (such as discrete differential DP and DP by successive approximation) [31, 66]. various gradient algorithms and Parikh's [50] linear-dynamic decomposition algorithm.

For larger systems (say, more than five or six reservoirs), the most successful modeling strategy still seems to be one which uses simulation, or a combination of optimization and simulation for deriving operating rules (for example, as illustrated in Figure 9), or one which uses models for real time operation. Each of these approaches can be aided by individuals having past experience in operating the particular multiple-reservoir system.

SOME ILLUSTRATIVE SYSTEMS AND ANALYSES

In this section a few representative North American multi-purpose multi-reservoir systems and their operating policy analyses will be discussed. These systems include

1) Severn-Trent-Rideau and Cataraqui River Basins in Ontario
2) The Great Lakes of Canada and the U.S.
3) Upper Delaware River in northeastern U.S.
4) Tennessee Valley Region in southeastern U.S.
5) Arkansas River Basin in southcentral U.S.
6) Central Valley Project in western U.S.
7) Columbia River Basin in northwestern U.S. and British Columbia.

Figure 12 indicates the general location of each of these reservoir systems in Canada or the U.S. The numbers indicating the particular river basins on the map correspond to the number indices in the above list.

FIGURE 12. LOCATION OF MULTI-RESERVOIR SYSTEMS
REVIEWED IN THIS PAPER

1. Severn-Trent-Rideau and Cataraqui River Basins in Ontario

Over the past several years, the multiple-reservoir operating policies of the Trent, Severn, Rideau and Cataraqui watersheds in southern Ontario, shown in Figure 13, have been studied using Acres general multi-purpose multi-reservoir simulation models [60,61]. A modified version of this model is being used as a dispatching aid for the day-to-day operation of the 58-reservoir Trent system. The improved system performance, compared to historical operating results, has been the main reason why the Trent-Severn Waterway Authority has continued to use this model for reservoir regulation.

The uses served in these watersheds include navitation, recreation, hydropower, flood control, water supply, fish and wildlife, and water quality. The operation of the multiple reservoirs in the basins is aimed at maximizing the collective benefits for all these water users. Because conflicts exist among various water users, and the relative importance of various uses will continually be changing, the operating strategy for those basins is flexible. For individual reservoirs, a combined rule curve, zoning and flow-ranging concept is used (see Figures 5 and 7).

For simulating operating responses for various operating strategies, a simulation model is used recursively from one time period to the next. In each time interval the optimal operating response is based on the current state of the system (reservoir levels and channel flows), the runoff forecast, and the prescribed operating policy. The optimization submodel uses an optimization out-of-kilter routine that minimizes the total penalty associated with any deviations from ideal conditions. The relative penalty coefficients are part of the input data and can be

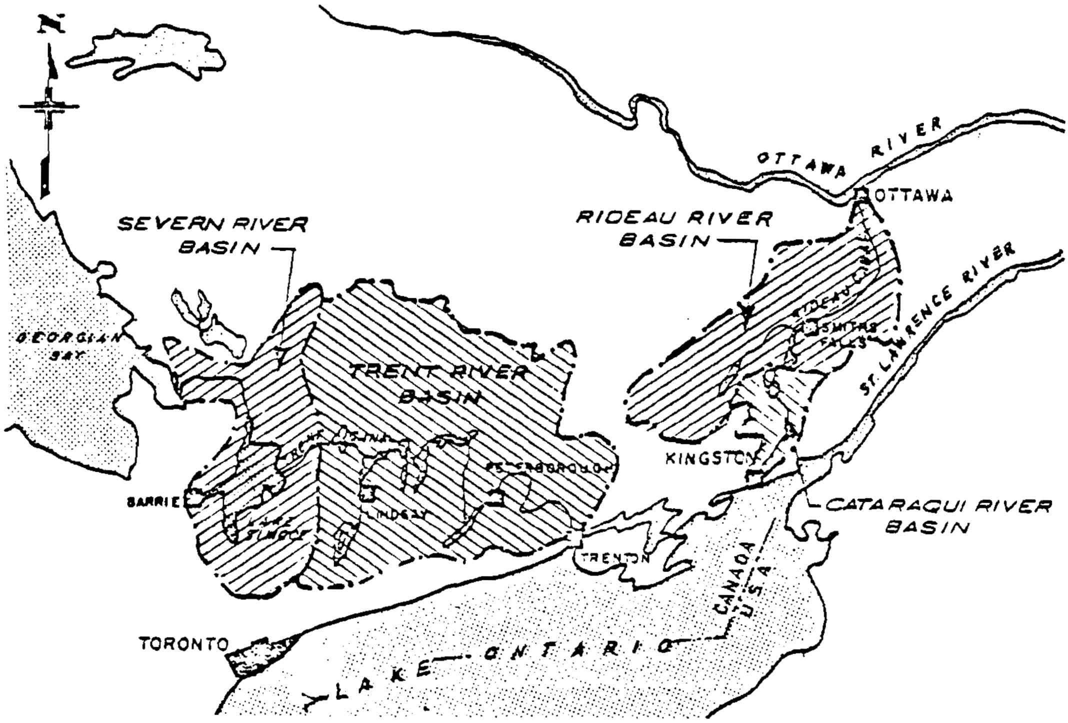


FIGURE 13. LOCATION OF TRENT, SEVERN, RIDEAU
AND CATARAQUI RIVER BASINS

altered at any time to reflect changes in operating policy. The model is also used to test the expected response of the system to changes in operating policy before implementing such changes.

To summarize, the simulation model is used for three purposes. It is used a) to examine alternative policies of system operation within the limits set by the individual reservoir rule curves, zones and channel flow ranges; b) to aid in the day-to-day operation of the multi-reservoir systems; and c) to aid in the water planning of improved operating rule curves, zones, and flow ranges, and water resource system structures.

2. The Great Lakes

The five Great Lakes ... Superior, Michigan, Huron, Erie, and Ontario ... with their connecting rivers and Lake St. Clair, have a water surface area of about 95,000 square miles (246,000 km^2). The total area of the Great Lakes basin, both land and water, above the easterly end of Lake Ontario is approximately 295,000 square miles (764,000 km^2). The Great Lakes Basin is shown in Figure 14.

Only two of the five Great Lakes are regulated. Lake Superior, the most western and upstream lake, has been regulated since 1921 (partially since 1916) in an effort to maintain lake levels within a specified range to enhance navigation and to reduce shoreline damage. The only other regulated lake, Lake Ontario, the most eastern and downstream lake, has been controlled since 1960. In view of the proportions of the physical quantities of water involved and the capacities of the channels connecting the lakes, complete regulation of these two lakes is not possible.

Water from Lake Superior is discharged into Lakes Michigan-Huron; that from Lakes Michigan-Huron into Lake Erie, and that from Lake Erie into Lake Ontario. Regulation of the outflow of any of the lakes of the system, other

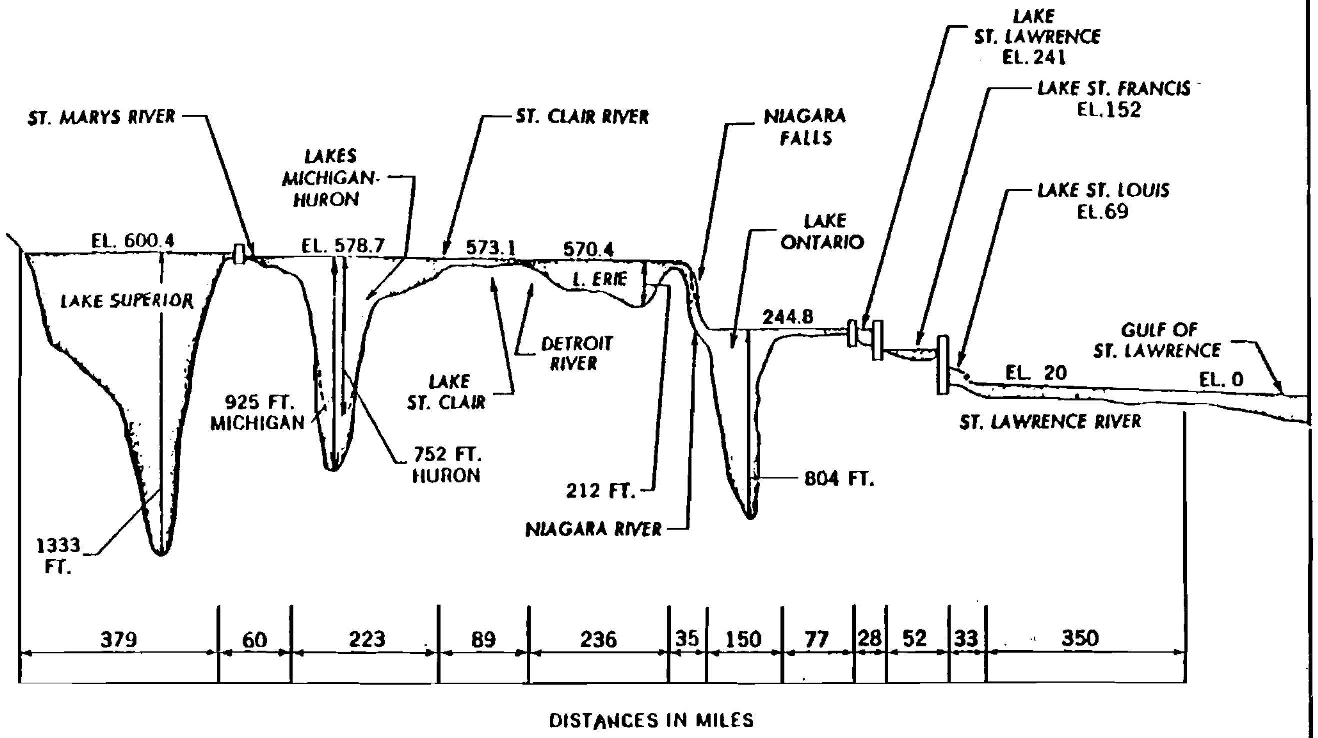
FIGURE 14. THE GREAT LAKES IN NORTH AMERICA

than Lake Ontario, affects the timing of flow into the lake immediately downstream, which in turn modifies the water supplies to the lakes situated further downstream. A profile of the system is shown in Figure 15.

Because of the broad and deep connection between Lakes Michigan and Huron, they have virtually the same level and are hydraulically considered as one lake. Because of the 20-foot (6 m) drop from Lake Superior to Lakes Michigan-Huron, the latter two lakes could be regulated without effect on the levels and outflows of Lake Superior. Outflows from Lakes Michigan-Huron are dependent on the levels of both Lake Huron and Lake Erie but to the greater extent on those of Lake Huron. Because these outflows are in part dependent on Lake Erie levels, control of the outflow of Lake Erie without control of the outflow of Lakes Michigan-Huron would affect the levels of Lakes Michigan-Huron. The regulation of the levels of Lake Erie would also significantly affect the timing of a major portion of the supply of water to Lake Ontario.

Lake Ontario outflow has been controlled since mid-1958 and the lake has been regulated since 1960. The regulation of Lake Ontario has no effect on the Lake Erie outflows because of the 326-foot (100 m) difference in level between these two lakes. About one-half of this difference occurs in the sheer drop at Niagara Falls.

The physical characteristics of Lake Ontario make complete regulation impossible. The inflows are unregulated, and the outflows can be partially controlled by two dams and some navigation locks. The objectives of regulation include a) maintaining a navigation system in the St. Lawrence River that permits deep draft ocean vessels to reach the Great Lakes, b) producing low cost hydroelectric power, and c) reducing the severity of extremely high and low lake levels to increase the benefits to shoreline property owners. To



ELEVATIONS OF THE LAKE SURFACES ARE AVERAGES EXPRESSED ON INTERNATIONAL GREAT LAKES DATUM (1955) AND ARE GIVEN TO THE NEAREST TENTH (1/10) FOOT. HORIZONTAL AND VERTICAL SCALES HAVE BEEN DISTORTED TO CONVEY VISUAL IMPRESSION.

FIGURE 15. PROFILE OF GREAT LAKES IN NORTH AMERICA

achieve an equitable distribution of the benefits from these potentially conflicting uses, some rules have been established that limit the extent of lake level variation and insure a specified minimum and maximum downstream discharge.

The current operating plan for Lake Superior is known as the "1955 Modified Rule of 1949," and is shown in Figure 16. This rule provides for monthly setting of the control works from 1 May to 1 December; alternations are made between 1 December and 30 April only when successive monthly mean stages of Lake Superior move from the intermediate stage range to the maximum or minimum stage range, or when successive monthly mean stages move from the maximum or minimum stage range to the intermediate stage range.

The present regulation plan for Lake Ontario, Plan 1958-D, provides for the weekly determination of water to be released through the various structures located in the St. Lawrence River. The regulated release is derived from a set of curves which show it as a function of the water level of the lake, and from a supply indicator which is an index of water supply conditions at the end of the preceding regulation period. There are two sets of curves for the year, one of which is shown in Figure 17. The release determined from curves is checked against a set of limitations on the flow release. If it is less than all of the maximum and greater than all of the minimum limitations, it is the flow to be released during the coming regulation period. If the release determined from the curves is outside of the range specified by limitations, the appropirate limitation flow is released.

The criteria for regulation of Lakes Superior and Ontario relate to the maximum and minimum water levels of the lake and to navigation depths. They are explicit and there is no element of operator judgment in their

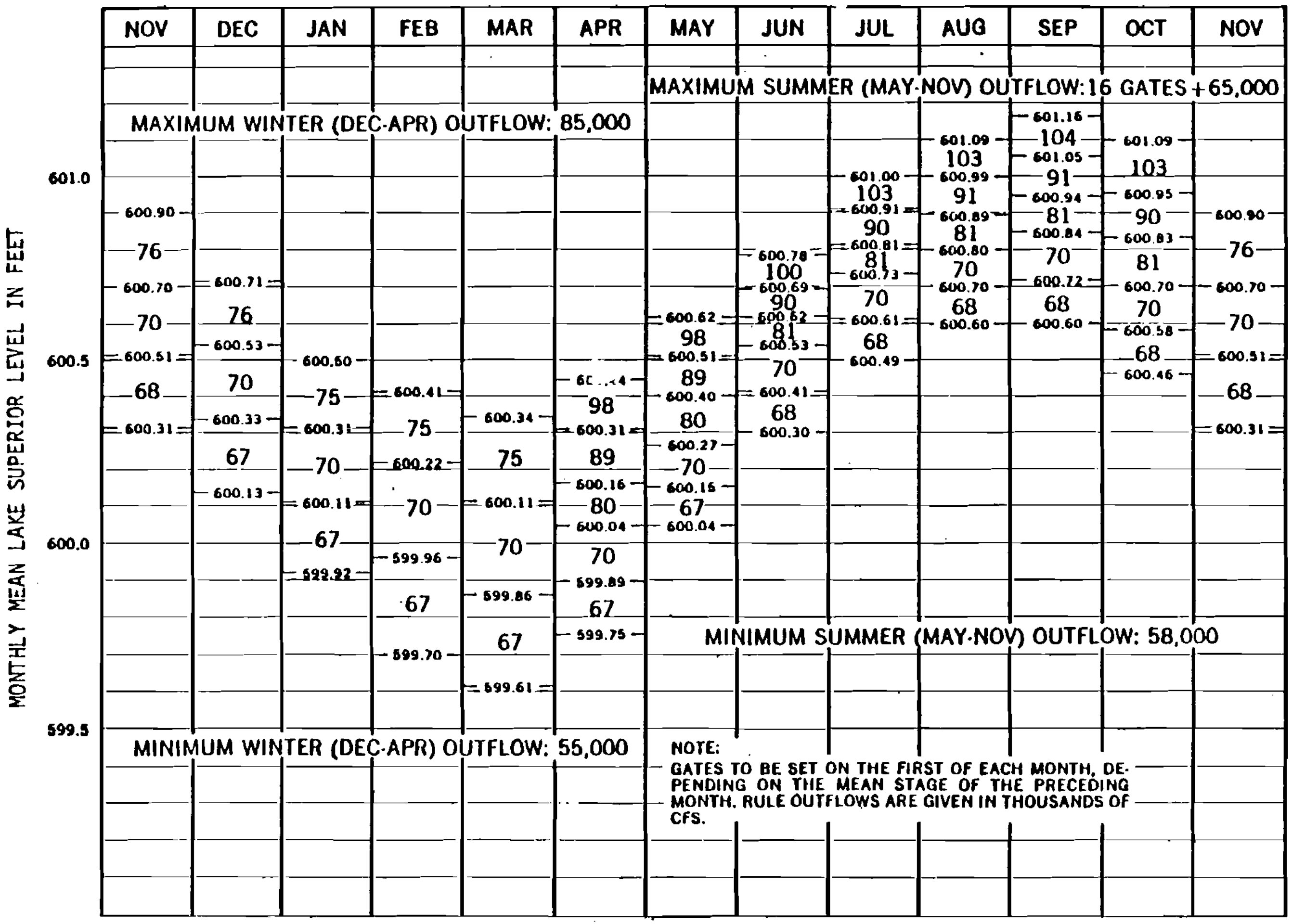


FIGURE 16. RELEASE RULE FOR LAKE SUPERIOR (1000 CFS = 28 m^3/s)

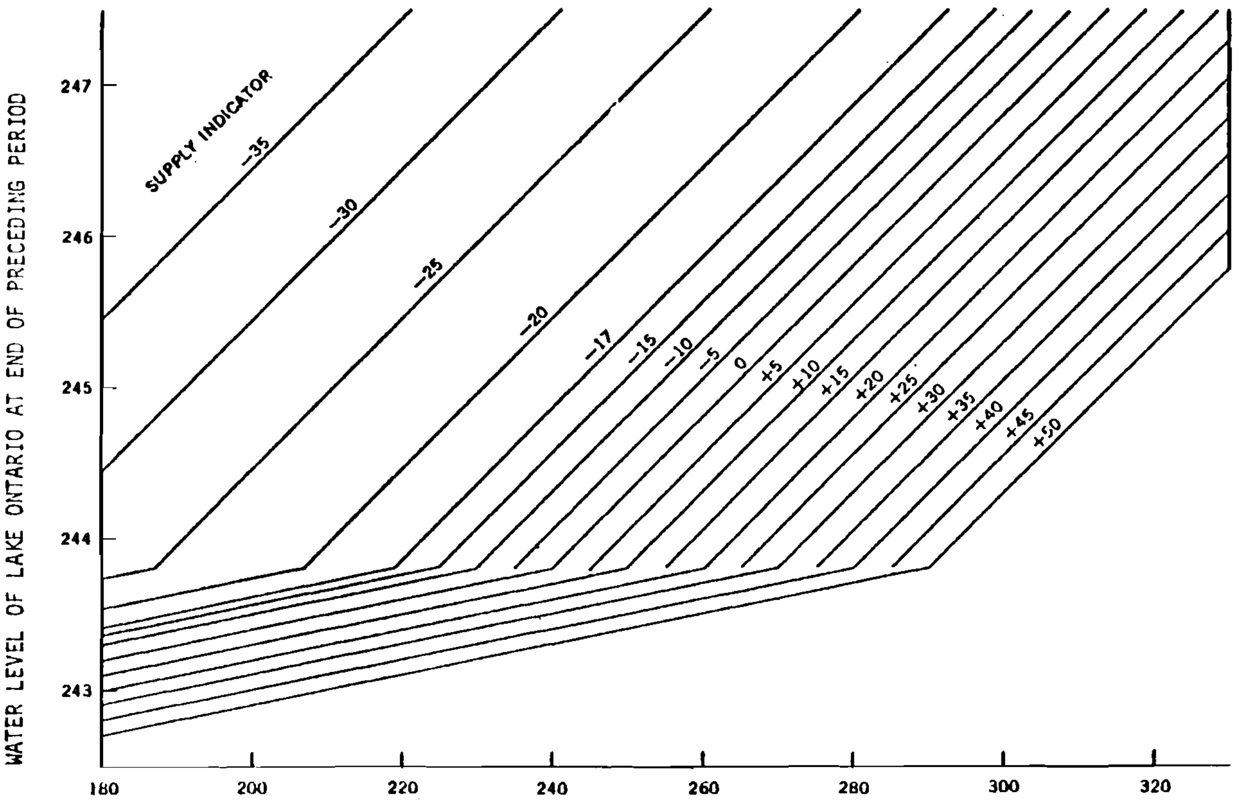


FIGURE 17. RELEASE RULE FOR LAKE ONTARIO, AUGUST-JANUARY INCLUSIVE, SUBJECT TO MAXIMUM AND MINIMUM LIMITATIONS (1000 CFS = 28 m^3/s).

interpretation, at least during periods of more or less normal supplies. The plans were designed by analyzing the available historical water supplies during critical supply sequences. Using these critical sequences as test data, tentative rules and limitations on releases of lake outflows were empirically determined, modified and tested until the resulting levels and flows gave the desired results. The plans were then tested using all available historical water-supply data to determine if any further modifications were necessary, and to ensure that the resulting levels and flows satisfied all criteria over the entire period.

The development of regulation plans in this fashion assures satisfaction of the specified criteria over the historical supply period, with resulting maximum benefits during critical periods, but does not result in the maximization of benefits over the entire period. In addition, successful operation of a plan developed using this approach is dependent in the future upon receiving water supplies no more critical, either in magnitude or sequence, than those used in the development of the plan. And indeed, during recent periods of record low inflows followed by record high inflows, the minimum and maximum levels and outflows could not be maintained. However, the lake levels that could be maintained were more moderate than would have occurred without regulation [18].

Current simulation studies are investigating the regulation of Lake Erie and the possible effects of diversions into and out of the Great Lakes Basin.

3. Upper Delaware River in Northeastern U.S.

The Upper Delaware River Basin contains three major reservoirs (Figure 18) that were developed by New York City as sources of water supply. Water is diverted from these reservoirs via underground tunnels to the city which is outside the basin. These reservoirs, supplying water to the city since 1953, 1955 and 1967, are operated in conjunction with two other major city water supply systems in two other river basins.

Diversion of water from the Delaware basin has become an increasingly controversial issue as demands increase for water downstream of these three reservoirs. The U.S. Supreme Court has prescribed certain operating requirements that force the city to release water from these reservoirs to meet certain minimum flow requirements downstream and hence reduce the reliability of that source of water for the city. New laws are being proposed to increase these releases and modify the current operation that causes rapid changes in river stages, flow velocities and temperatures. New York City and New York State are presently evaluating various reservoir operating policies in an effort to derive operating policies that can best satisfy the riparian water users and still provide a sufficiently reliable source of water for the city.

Using a monthly simulation model and the record of historical monthly flows, alternative diversion and release rules of the form shown in Figure 19 are being evaluated. These rules are defined for the three combined reservoirs, and are used together with level indices and level number storage balancing functions (Figure 20) that indicate the desired combinations of reservoir levels in the three reservoirs [48]. It remains to be seen whether or not the results of these simulation studies will lead to a satisfactory agreement between the interested parties in this conflict.

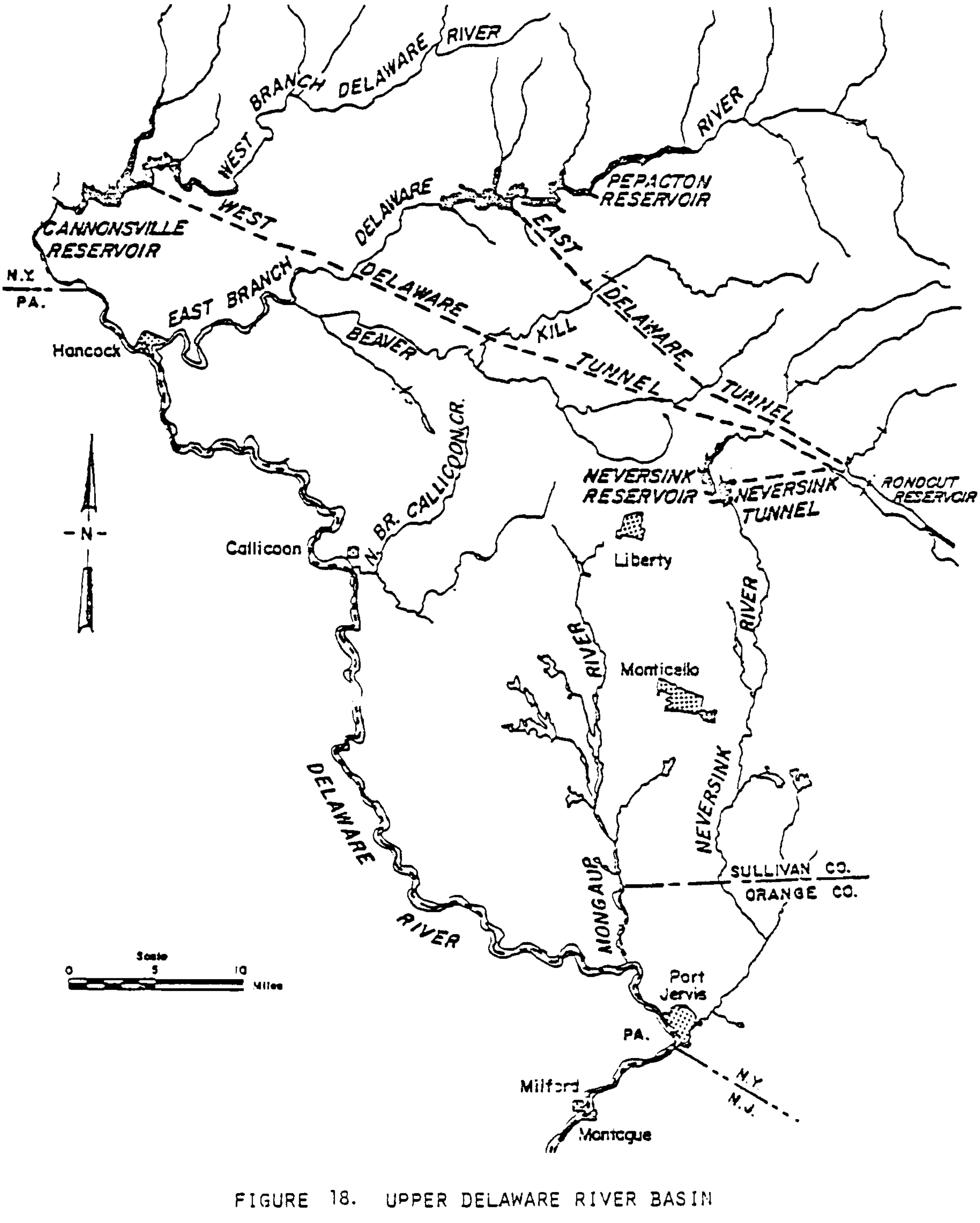


FIGURE 18. UPPER DELAWARE RIVER BASIN

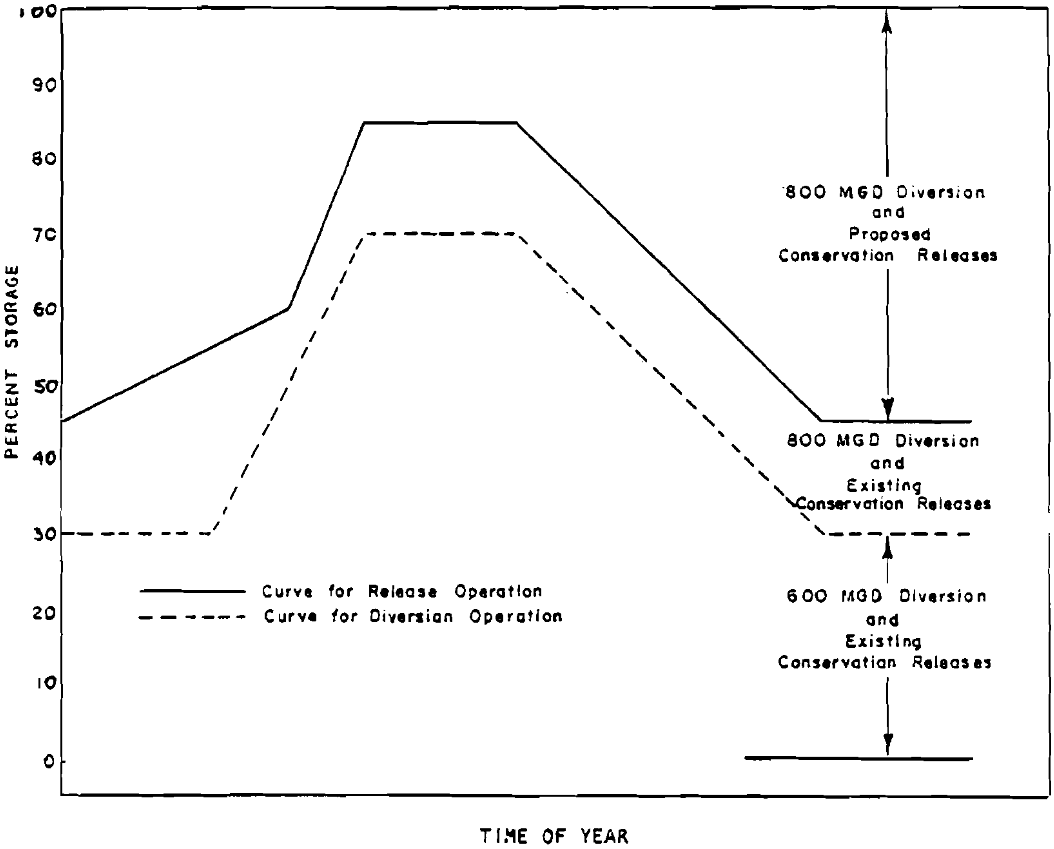
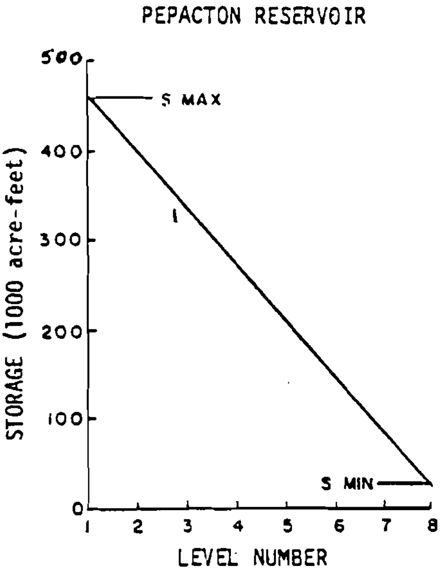
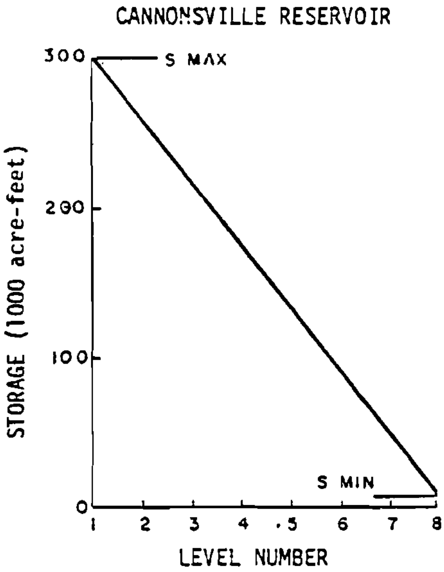


FIGURE 19. RULES FOR DIVERSION AND RELEASE OPERATIONS FOR UPPER DELAWARE RIVER RESERVOIRS ($1\ MGD = 0.0438\ m^3/s$)





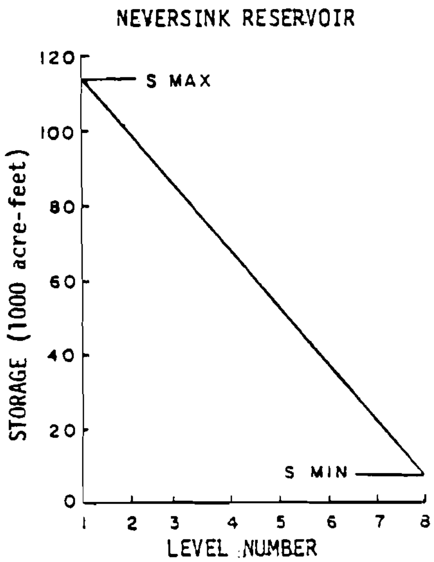


FIGURE 20. RESERVOIR LEVEL NUMBER - STORAGE CURVES FOR UPPER DELAWARE RIVER RESERVOIRS

4. Tennessee Valley Region in Southeastern U.S.

The TVA system includes 21 multiple-purpose reservoirs and 14 single-purpose hydropower reservoirs. In addition to these 35 reservoirs, 8 reservoirs operated by the Corps of Engineers and used for generating hydroelectric energy, are part of the overall 43 reservoir system shown in Figure 21. The primary purpose of TVA reservoir system operation is streamflow regulation for navigation and flood control, and as consistent with these purposes, for the generation of hydropower. Also, it is a TVA policy to maintain as high water quality as possible in all streams and reservoirs and to provide recreation and water supply as consistent with the primary purposes. The scheduling of the reservoir system is presently guided by flood control limits, regulating zones, normal maximum pool elevations and normal minimum levels; normal ranges of pool levels; balancing of storage volumes between reservoirs; economy rule curves for hydropower use; power demands; and hydrologic conditions [35].

The flood control limits and regulating zones are used during the winter flood season (January, February and March). The reservoir levels are usually kept below these limits or within the regulating zone. During the summer, they are generally kept below normal maximum pool levels. Above these limits, reservoir storage is reserved for temporary flood storage. Within and below the regulating zone, discharges from the reservoirs are scheduled to serve power needs. The normal ranges of pool levels were established by studies and experience. They occupy a zone below the flood limits and the normal maximum pool levels and are used by power operations to schedule hydro loads and to keep reservoir storages in a system-wide balance.

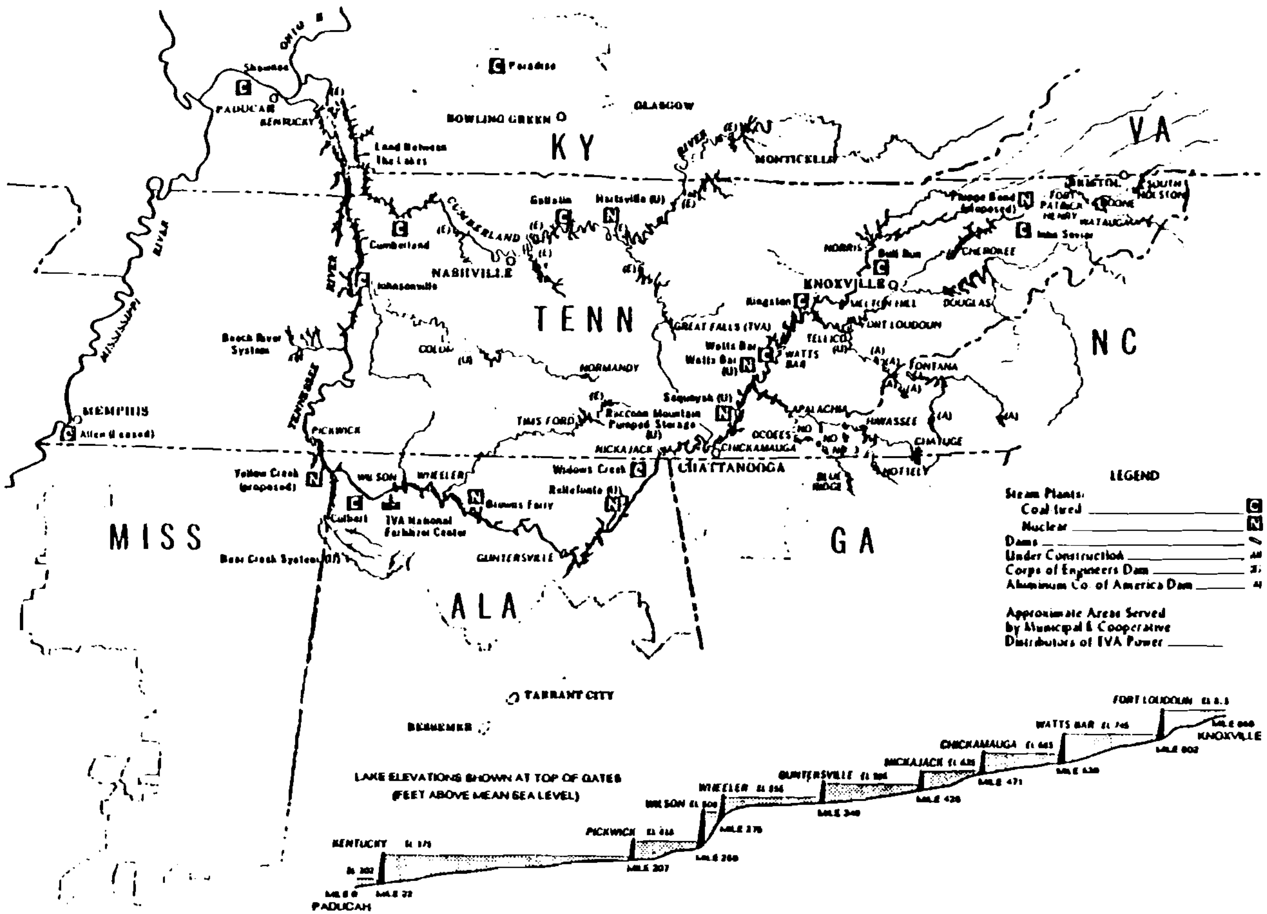


FIGURE 21. TENNESSEE VALLEY AUTHORITY SYSTEM AND PROFILE OF TENNESSEE RIVER RESERVOIRS

The main river reservoirs follow rather closely a fixed "normal operation" guide (prescribed water levels). The annual range of water level fluctuations is relatively small in these reservoirs as compared to tributary storage reservoirs.

As an example of an annual operation plan, Figure 22 shows a typical operating guide curve for a tributary multiple-purpose reservoir providing flood-control storage and conservation storage for power and navigation. The storage reservation for flood control on March 15 was determined as the amount necessary, in conjunction with other reservoirs and levees, for controlling the maximum probable flood at Chattanooga, a critical downstream location. The greater flood-storage reservation on January 1 gives assurance that the March 15 reservation will be available in event a series of floods makes it difficult to draw down the reservoir to the March 15 level. Drawdown of the reservoir prior to January 1 provides useful water for meeting navigation and power production requirements during the earlier drier months, and normally can be accomplished with greater assurance and efficiency than would be possible during the January 1-March 15 period. The lesser reservation on March 15 and thereafter makes allowance for the decreased chance of floods near the end of the Valley-wide flood season. After March 15, the reservoir is allowed to fill more rapidly dependent upon hydrologic conditions, and may be filled to normal maximum level if rainfall is abundant. Deficient rainfall, combined with heavy demands for hydroelectric power production during the normal filling period, April 1 to June 1, will prevent filling of the reservoir, which then may remain substantially below top level through the summer. A small amount of flood detention capacity is reserved through the summer months as a protection against flood-producing storms over limited areas.

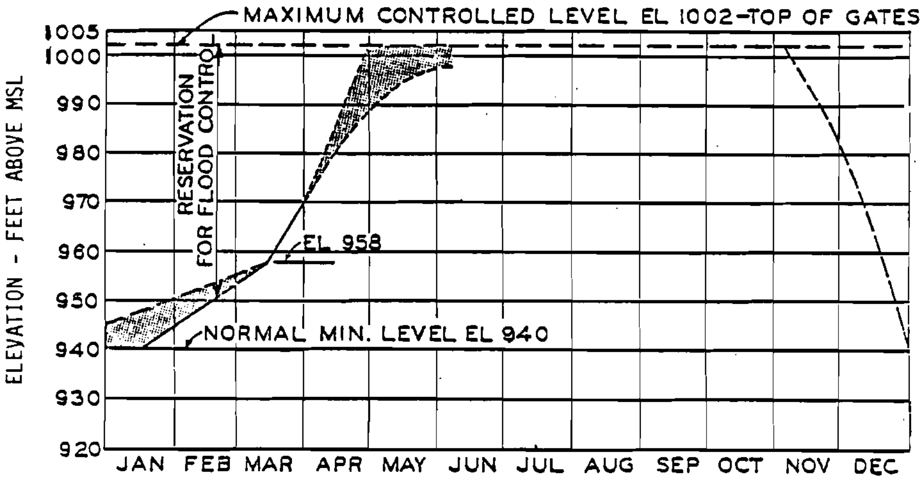


FIGURE 22. TYPICAL OPERATING GUIDE CURVE FOR A TVA TRIBUTARY MULTIPLE-PURPOSE RESERVOIR

When heavy runoff occurs during the flood season, discharge from the dam is reduced or cut off and the reservoir may be temporarily filled above the operating guide curve, thus storing flood waters and reducing downstream flood crests. When flood danger has passed, the reservoir is returned to seasonal level by releasing water at rates that will not create or supplement downstream flooding. Sometimes this drawdown can be accomplished by operating the hydroelectric plant at turbine capacity until the necessary quantity of water has been discharged from the reservoir. Often, however, it is necessary to release additional water through sluiceways or spillways to lower the reservoir level more quickly and regain the detention space needed for future rains. Spilling of this water is proof that TVA places priority for flood control over that for power -- a definite stipulation in the TVA Act.

Lowering of the reservoirs to prepare the system for the next flood season normally begins in early summer and accelerates during the relatively dry fall months. The water is withdrawn gradually, to supplement diminishing natural streamflow, for navigation improvement and power production. By late December, the reservoirs normally have been returned to low levels, completing the annual cycle, as shown by Figure 22.

An example of an annual operation plan for a multi-purpose main Tennessee River reservoir, which also provides flood control storage and conservation storage for power and navigation, is shown in Figure 23. In addition to conservation storage, it provides a permanent pool for navigation. The minimum pool, elevation 675, was determined by the specified navigation depth at critical points in the reservoir, and the maximum pool, elevation 685.44, was determined by reservoir limitations and the location of the next upstream dam site. Flood control or conservation storage therefore was limited to the zone between these two levels, but during the usual Valley-wide flood season the full amount was reserved for flood control, except for minor fluctuations due to turbine operation. In order to retain storage capacity

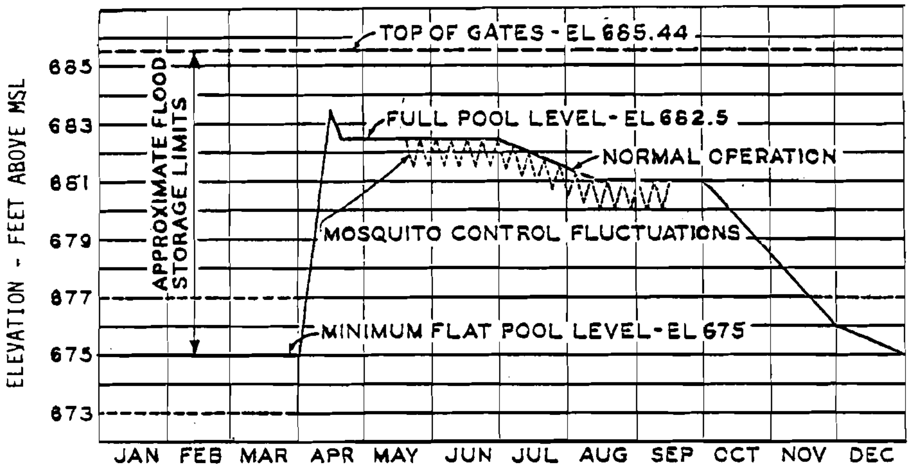


FIGURE 23. TYPICAL OPERATING GUIDE CURVE FOR A MAIN TENNESSEE RIVER MULTIPLE-PURPOSE RESERVOIR

for flood control, drawdown to elevation 673 at the dam may be permitted. After March the reservoir is filled to elevation 682.5 and the zone between elevation 682.5 and elevation 685.44 is the minimum reservation for flood storage during the summer.

The fluctuating dashed lines show 1 foot (0.3 m) weekly changes in level for control of lake-breeding mosquitoes. These planned fluctuations of main river reservoirs usually begin in June and continue into September and are part of a yearly cycle of water level management. The main river reservoirs are fluctuated in tandem throughout the reservoir chain.

The 1 foot rise above elevation 682.5 shown on Figure 23 (about the middle of April) is a surcharge of the reservoir above normal summer level to strand drift and debris brought into the reservoir by winter floods. After the reservoir has been surcharged for about 24 hours, the level is drawn back to normal summer level within one day. Much of the floating driftwood and debris is stranded on the shoreline above the water level of the reservoir. This operation serves as a means of cleaning the reservoirs, thus reduces the hazards to recreational boaters and water ski enthusiasts, reduces the production of mosquitoes, and improves the aesthetic appearance.

The operating guides for the main Tennessee River reservoirs also require the lowest reservoir levels during January, but unlike the tributary reservoirs, available flood storage space is so small that low levels are held until near the end of the flood season before filling to summer levels. Reservoir levels provide channel depths adequate for navigation throughout the year. During a flood control operation, the main river reservoirs may be temporarily filled to top-of-gates level, if required, thus storing flood waters and reducing downstream flood crests. As flood danger subsides, the reservoirs are promptly returned to seasonal levels by releasing water at rates that will not release excess water from the main river reservoirs through the spillways to lower the reservoir level more quickly and regain the detention space needed for future rains.

Lowering of the main Tennessee River reservoirs also begins during the summer and accelerates during the relatively dry fall months, thus pulling the water level away from the encroaching vegetation and preparing the system for the next flood season. The water is withdrawn gradually for navigation improvement and power production. By late December, these reservoirs also have been returned to low levels, completing the annual cycle, as shown by Figure 17.

During normal flow conditions, the primary operation objective is economic power generation. The navigation objective is satisfied by maintaining water levels in the main river reservoirs at or above minimum pool levels. Flood control is satisfied by maintaining proper reservoir levels. Daily reservoir scheduling is based on projected hydropower loads and a partial hydroplant preschedule by the Office of Power. These projections are checked, modified as necessary, completed and approved by the River Management Branch. By agreement with the Corps of Engineers, the Cumberland system is also operated in this way. During flood operations, the River Management Branch and the Corps of Engineers control their respective systems. The Ohio River Division of the Corps of Engineers guides TVA's Kentucky Reservoir operation in order to reduce flood stages on the lower Ohio and Mississippi Rivers. At times, Kentucky and Barkley Dams have been shut off completely thereby storing the entire Tennessee and Cumberland River flows in order to reduce the flood stages on the lower Ohio and Mississippi Rivers.

A project to develop some mathematical water resource management models began in 1971. Its purpose is to provide the manager of the reservoir systems with an assessment of the impacts of various operating policies on flood control, navigation, power generation, water quality and recreation. The project is expected to enhance existing methods by providing more comprehensive information for all essential operating purposes faster than is presently possible. The proposed methods should enable the water manager to cope with the steadily increasing complexity of day-to-day reservoir operation and long-range planning. This increasing complexity is caused by the increased attention that is being paid to the interaction between physical, economical and environmental factors and by more diversified public interest in water and reservoir use. To the extent possible, the methods will use quantitative measures of effectiveness to assess the relative merits of alternate decisions.

The project includes the development of mathematical models for the various aspects of the operation problem. They comprise the prediction of system inputs and demands, the simulation of the physical characteristics and the flow, quality and other processes going on in the reservoir system, the evaluation of operating objectives and the search for optimal operating strategies.

The project is subdivided into four major segments: two weekly planning and operation models and a daily and an hourly operation model. To date (1979), a first weekly planning model has been completed. A second weekly planning model is in an advanced stage of development. Also, elements of the daily and hourly planning and operation models are under development.

The weekly planning models provide a computational tool for systematic analysis of the TVA reservoir system for planning and operational studies by weekly time steps. However, they consider in a simplified way also the transient phenomena within the week that have consistent effects on the results, such as flood peaks, peak and off-peak hydro and thermal generation and the impact of transient flow on water quality. The first weekly model evaluates, for a given system configuration, power loads and hydrologic inputs, the cost of navigation, flood control, power production, water quality management and recreation and finds operation projections (reservoir level sequences) for 18 storage reservoirs over time horizons of up to 52 weeks that minimize a specified performance index within all specified constraints on water levels, flows, etc. Presently the performance index (or composite objective function) is expressed as the sum of weighted costs associated with the five objectives. Dynamic programming by successive approximations is used to minimize the performance index.

The second weekly model is an enhanced version of the first. Its principal feature is the use of a stochastic dynamic programming approach to the long-range guide calculation using a dimensionally-reduced system and a priority ordering of constraints. Weekly optimization is performed by a non-linear programming technique. Presently, in this model, only power generation costs and flood damage costs are considered. Feasible and optimal operating policies are found subject to all operating constraints ordered by prespecified priority.

Various models to be used in daily reservoir operations are also under development. A dynamic flow routing model for the upper half of the main river cascade by hourly time steps is nearing completion. This model will be used for flood control and water quality planning. Also underway is a program to enhance daily and hourly streamflow forecasting techniques. Conceptual and statistical techniques are under investigation. A daily scheduling model for daily and/or hourly time steps is in the early planning stages. The hourly model will be used in operations when short-time step considerations are important, as in flood control operations. Gradual implementation of all models is planned to be completed by 1985.

5. The Arkansas Basin in Southcentral U.S.

The Arkansas River Basin, a portion of which is shown in Figure 24, has recently been studied in an effort to improve the operation of the 16 reservoirs shown in Figure 24 [13]. These reservoirs are operated by the U.S. Army Corps of Engineers to meet demands for water supply and low flow augmentation, hydropower, flood control, navigation and recreation and wildlife enhancement. Simulation models together with 34 years of historical monthly flows were used to evaluate the impacts of various operating policies, defined by storage zones similar to those shown in Figure 25 for the equivalent percent of basin storage utilized upstream of Van Buren, and by reservoir level balance curves, similar to those in Figure 26. The equivalent percent of basin storage utilized is the total existing upstream reservoir storage utilized plus the predicted inflow in excess of the predicted releases for the next 5-day period divided by the total upstream reservoir storage capacity. Reservoir balance curves define the priority in which the water levels in reservoirs are drawn down to evacuate flood storage and meet downstream flow requirements. Reservoir levels in each reservoir are indexed and reservoir releases from upstream reservoirs are to be made so that each reservoir is

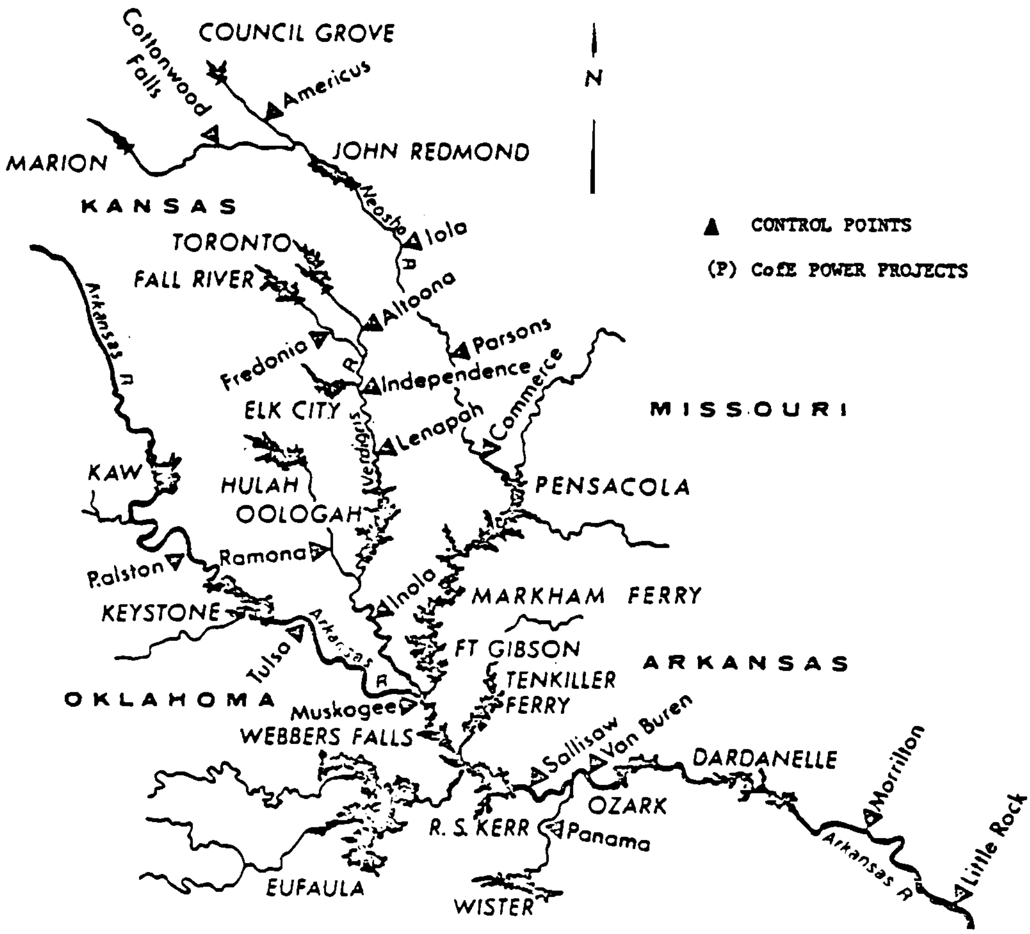


FIGURE 24. ARKANSAS RIVER RESERVOIR SYSTEM

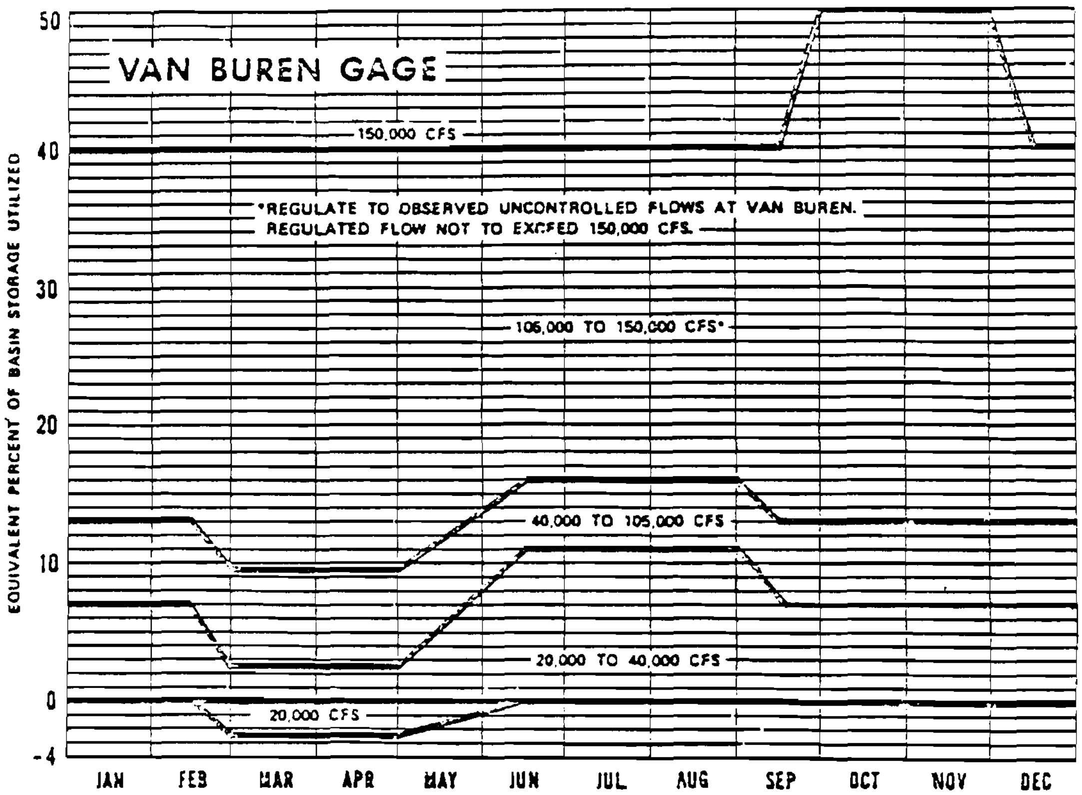


FIGURE 25. RESERVOIR STORAGE ZONES FOR FLOW REGULATION AT VAN BUREN

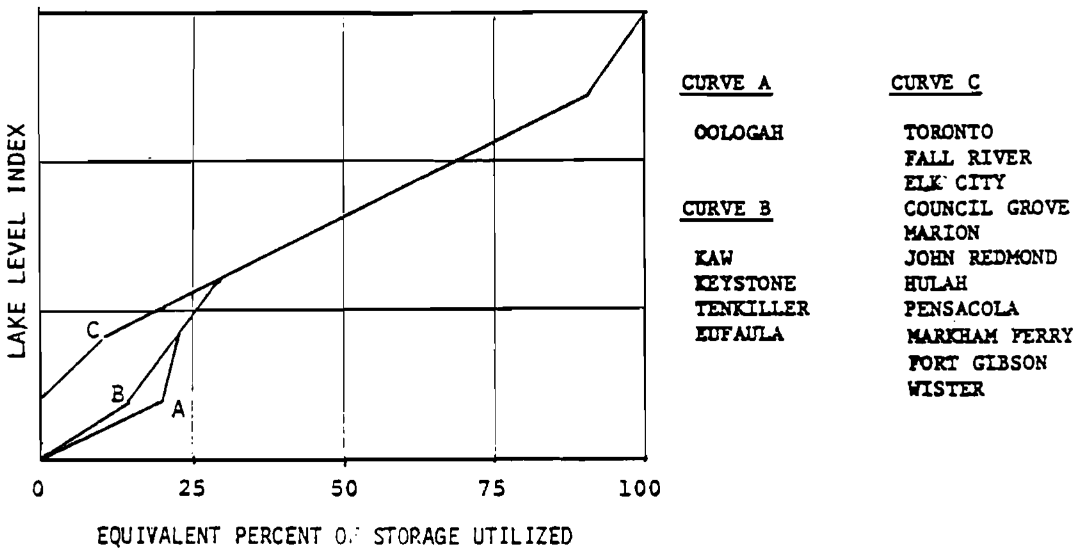


FIGURE 26. LEVEL BALANCE CURVES INDICATING RESERVOIR RELEASE PRIORITY

at its appropriate level index, if possible, as defined by Figure 26. As shown in Figure 26, when the equivalent storage volume is above 30 percent, all projects are given equal priority with regard to reservoir releases and to the use of available channel capacity. This is based on the assumption that the probability of filling each reservoir above this volume is about the same.

The results of numerous simulation runs were compared and evaluated based on flow magnitudes and reliabilities. The rules for the selected plan are in fact what is shown in Figures 25 and 26. The regulated flow targets are dependent on the time of year and on the equivalent upstream storage. However, studies of reservoir operation in this basin continue, and as use priorities change, undoubtedly so will the operating policies.

6. Central Valley Project in Western U.S.

The Central Valley Project is a multi-purpose multi-reservoir project located in the Central Valley of California. Since its authorization in 1935 it has constantly grown in terms of project facilities and water demands. A schematic of the Central Valley Project System is shown in Figure 27 [45]. The complexity of the system and the growth of demands on the system, along with advances in systems analysis methods, recently led the Bureau of Reclamation to initiate the development of water forecasting models.

Since 1970, mathematical models for the CVP have been developed in three general areas:

a) System operation models to provide operational decision-making information;
b) Water quality models;
c) Hydrologic models that simulate all significant components of the hydrologic cycle.

Of interest here are the system operation models for the multiple reservoirs and associated canals, pumping stations and power plants shown in Figure 27.

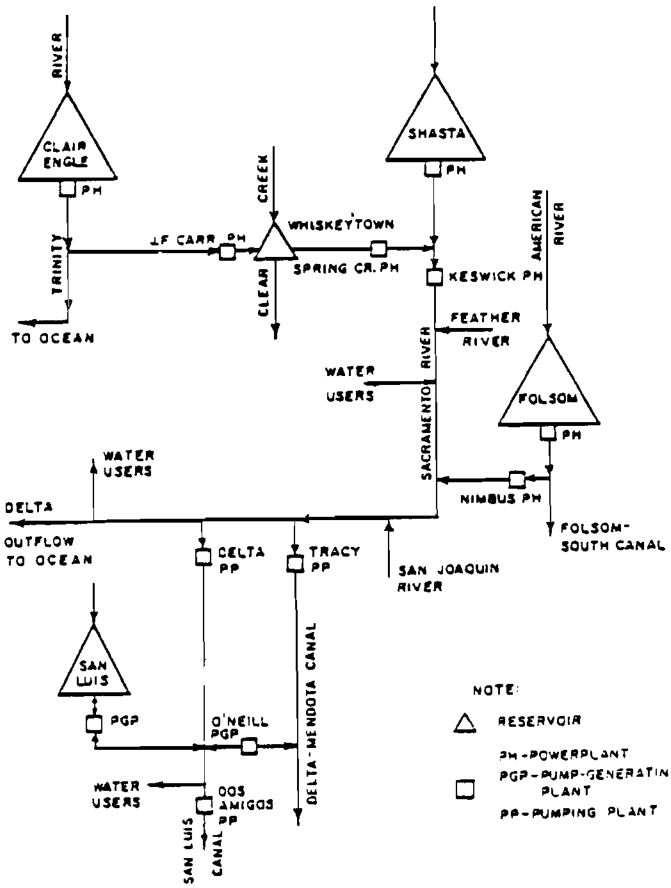


FIGURE 27. THE CENTRAL VALLEY PROJECT

While the reservoirs are operated to satisfy multiple purposes, the operating models are designed to maximize energy generation subject to minimum acceptable levels of other objectives. Using this approach, six models have been developed to provide CVP operators with tools to improve their decisions. Not all of these modeling efforts have been successful.

Shasta-Trinity Pilot Model

The first model developed was a daily model for the Shasta-Trinity portion of the project. This pilot model utilized a state incremental dynamic programming method to maximize the portion of the CVP's firm energy output from Shasta and Clair Engle Reservoirs, while satisfying the other objectives as constraints. This initial two-reservoir modeling effort convinced the CVP managers that the concept of modeling the project to provide decision-making information was valid. On the basis of the accomplishments in the development of this model, the managers authorized the development of three interrelated models - a monthly model which would cover a 12-month period, a daily model which would cover up to 31 days, and an hourly model which would cover one day. Each model would be a separate computer program providing data for the succeeding model.

USBR Monthly Optimization Model

The initial monthly model was developed by the Bureau of Reclamation and used incremental dynamic programming to determine the forecasted operation of the four major reservoirs in the CVP system. During the initial stages of implementing the USBR Monthly Optimization Model, the program functioned as designed - to maximize the energy generation within the normal operational constraints of the CVP. Each month the results from the model were evaluated and then used to produce an operation forecast report for the following 12 months.

As the model gained acceptance, it became evident that with some program modification to produce a report, the output from the model could be used directly as the forecast of operations. This system worked well during the wet years of 1973-75 but developed problems with the beginning of the 1976-77 drought.

As the reservoir levels dropped, power curves that had been developed for the model were extrapolated beyond the range of data used to derive them and found to be inaccurate in the lower reservoir ranges. Hard constraints that had been programmed into the logic, such as required minimum releases from the reservoirs to the streams, required changing as the water supply decreased.

As required changes and modifications were included, the program became larger and required longer periods of computer time. While working under a time constraint, this became more and more of a problem and was finally the determining factor in returning to doing the forecast manually.

CVP Monthly Simulation Model

As a direct result of the 1976-77 California drought and the problems with the USBR Monthly Optimization Model, the CVP Simulation Model was developed. The model was designed to simulate the monthly operation of the CVP over a 12-month period.

During the drought it was discovered that many of the fixed constraints in the USBR Monthly Optimization Model were actually flexible under certain conditions and could be manipulated. Therefore, the simulation model was designed to accept special exceptions to normal operating procedures. This capability is now considered important even in normal years, because the demands on the CVP have increased considerably since the development of the optimization models.

The CVP Simulation Model is used to produce a 12-month forecast of reservoir and power plant operations. First, a monthly strategy is determined which meets all minimum water requirements, i.e. contracts, mandatory releases, flood control and pumping demands. Then, the user has the freedom to interactively modify reservoir releases and pumping requirements to produce an acceptable energy generation. These steps are repeated for each month of the forecast period. Finally, the model may be rerun any number of times with different input data and/or different operating criteria. This process allows the user to examine alternative operating strategies and evaluate the benefits and trade-offs associated with each.

The model is programmed in FORTRAN. It requires a relatively small amount of core storage which allows it to run interactively. With user interaction, the model can be run in approximately 1 hour of clock time. The program actually executes in about 3 seconds of computer time, which allows it to be run repeatedly. Other factors which affect the model's usefulness are its ability to handle many options and include unusual circumstances. The human intervention aspect also contributes to a high level of confidence in model results which might not otherwise exist.

In summary, the CVP simulation model provides an effective means to search for the best solution to increasingly difficult and interrelated problems associated with the operation of a large multi-purpose water resources project.

UCLA Monthly Optimization Model

Althought the incremental dynamic programming technique used in the USBR Monthly Model was successful when applied to the four-reservoir system, the addition of two more reservoirs would cause the program to exceed the capabilities of the available computers. This problem was identified in 1973. At that time, the Bureau of Reclamation and the Office of Water Resources Research (now OWRT) contracted with the University of California at Los Angeles (UCLA) to develop a procedure to alleviate the dimensionality problems [72]. The contract called for the development of all three models, monthly, daily and hourly. The UCLA monthly model used a combined linear programming-dynamic programming (LP-DP) procedure to optimize the system. This procedure results in less required computer storage and faster running times than the USBR Model. The advantages of this procedure include the relatively easy method of changing or adding constraints and the addition of reservoirs.

UCLA Daily Model

The methodology used in the Daily Model is similar to the monthly model except that use of the LP-DP technique is unnecessary. This is because a sufficiently accurate result for practical purposes can be obtained by using the same value of energy delivery constraint each day of the month since hydroelectric head changes from day to day are small and power surge constraints limit flow variations. The number of constraints increases from 54 (monthly) to 70 (daily). These additional constraints define lag times of water deliveries and storage requirements of the regulatory reservoirs.

The main problem with the daily model for the user is the vast amount of input. Further, the implementation of the daily model began in 1976, which was the beginning of a 2-year drought. Due to the low water quantities, the daily model became unusable. Therefore, priorities changed and further implementation was ceased.

UCLA Hourly Model

The hourly optimization model is a two-phase procedure for operating the CVP over a 24-hour period. The program maximizes the operation of CVP power facilities while meeting water constraints. Phase I determines a good feasible operation through an iterated linear programming process. Phase II uses an incremental dynamic programming, successive approximations process, applied to the Phase I policy to arrive at an optimal hourly schedule of water releases at CVP facilities. Constraints on the system include minimum and maximum releases from the CVP reservoirs and an hourly power demand from a major contractor for CVP power. Output from the model includes hourly release schedules for each reservoir and hourly energy generation from each facility. The program also schedules the most efficient number of units to use at each powerhouse.

Several problems remain with the Hourly Model. Each problem is responsible, in part, for each of the other problems. Currently the scheduling of units at the power facilities sometimes results in multiple startup and shutdown of generators during a 24-hour period. Even though this scheme of operation may be optimal for power production, in actuality the operation is unrealistic due to the strain placed on equipment. It is possible to place additional constraints on the program to provide acceptable scheduling, but this in turn aggravates a problem of program execution on the computer.

Currently the hourly model is running on the USBR computer system but is not being used as an operational tool because of excessive running times. Development is still being done on methods to improve turnaround time and unit scheduling. Other proposals, such as adapting the program to a local minicomputer control system, utilizing only portions of the program, and integrating portions of the program with existing inhouse programs, are being discussed.

Table 1 summarizes the current status of, and future plans for, these models [45].

MODEL	STATUS	FUTURE PLANS
Shasta-Trinity	Superceded.	None.
USBR Monthly Optimization	Retired.	None.
CVP Monthly Simulation	Operational.	Will be expanded to include new facilities and used in conjunction with UCLA monthly model.
UCLA Monthly Optimization	Currently inactive. Requires revisions to power curves and some slight logic changes.	Will be revised and expanded to include new facilities. Will be used in conjunction with the simulation model.
UCLA Daily	Currently inactive.	No immediate plans to activate.
UCLA Hourly	Currently being reprogrammed to run on USBR CYBER system. Not yet operational.	Will expand to include new facilities and adopt the model to the local mini-computer control system.

TABLE 1. CURRENT STATUS AND FUTURE PLANS FOR MODEL DEVELOPMENT AND USE FOR THE CENTRAL VALLEY PROJECT (1979).

Eight years were spent in the development and use of operational CVP models. This experience has led those involved to three major conclusions.

1. Since the decision was made to model a major portion of the CVP, the average generation per acre-foot of water has increased about 10 percent. While some of this increase might be attributable to the use of the models, most of the increased generation occurred during the development stages. The knowledge of the system gained by the operators during the process of developing the model is more responsible for the increased generation than the actual models use. In the future, as the personnel in the operations office change, the continued use of the models will provide a method for the new operators to quickly acquire the knowledge required to make sound operational decisions.

2. The interactive simulation model used in conjunction with an optimization model is a very effective method of examining much of the information available to an operator. Optimization Models are decision-making tools and not the producers of a final operational strategy. This fact was temporarily forgotten when the USBR Monthly Optimization Model was modified to produce a final operation report. Although the generation of energy is an important project objective it is not the only objective. Other project objectives cannot always be formulated into equations that are static.

3. Most water resources project operators are not operations research analysts. Therefore, if models are going to be accepted and used by the people that make the decisions, the model cannot be so complex that the user has difficulty in operating and maintaining it.

7. <u>Columbia River Basin in Northwestern U.S. and British Columbia</u>

The Columbia, Figure 28, is the fourth largest river in North America and the nineteenth largest in the world. It is an international river in western North America flowing from Canada into the U.S. that has been highly developed for multiple-purpose use by a system of over two hundred reservoirs under diverse ownership. Hydropower is one of these important functions. The reservoir system includes over fifty hydroelectric projects that provide approximately three-fourths of the region's electrical power. One-half of all U.S. hydropower is in this region.

The Columbia River has been developed and is operated for multiple water-resource purposes including not only hydropower but also irrigation, navigation, flood control, fish and wildlife, recreation, municipal and industrial water supply, and water quality.

A brief review of the types of dams and reservoirs and their roles in the Columbia System may help to understand how the system is managed. Large headwater storage reservoirs fill during high flow periods, thereby reducing floods, and then later release extra water for power and other purposes. The "annual storage" reservoirs in the Columbia System are usually emptied and refilled once every year. They can be filled each year even if drawn to the bottom in a low flow year. The "cyclical storage" reservoirs are also emptied and filled on a yearly basis but if drawn to the bottom, they will not completely refill during low flow years. Measurements of the mountain snowpack are used as an index to forecast runoff which is used to determine rule curves which

FIGURE 28. THE COLUMBIA RIVER BASIN SHOWING MAJOR RESERVOIRS

indicate how much reservoir space can and should be evacuated in advance of the snowmelt runoff and still completely refill with reasonable confidence.

There are more than 46 million acre feet (MAF) (1 MAF = 1234 x $10^6 m^3$) of active storage in the Columbia reservoir system but less than 43 MAF are directly used for power production. Most of this is in the 15 largest reservoirs that include more than 40 MAF. The remaining storage, in the smaller reservoirs, is less controllable on a system basis. Of all the dam and reservoir projects in the Columbia Basin, approximately 100 are involved in power production but most of the power is produced by about half of these. There are additional projects outside the basin that contribute to the region's power. The Army Corps of Engineers operates 21 reservoir projects in the Pacific Northwest (20 in the basin and one outside) that produce 40 percent of the region's hydroelectric power. The Bureau of Reclamation operates 9 projects (all in the basin) that produce about 20 percent while the remaining 40 percent of the hydroelectric power is produced by several public (city and county) agencies and private utilities. There are also a few large and several small thermal powerplants in the region.

No single agency or interest group controls the Columbia River. Interagency cooperation is a necessity for responsible management of the Columbia, which rests upon an intricate formal and informal "check and balance" system. The Army Corps of Engineers is the largest operator of reservoir projects in the region and the Bureau of Reclamation is the second largest. Both of these agencies operate their reservoir projects for multiple purposes. The Bonneville Power Administration is the largest operator of transmission lines in the region, and it is the marketing agent for the power produced by the Corps and Bureau dams. Many other dams in the U.S. portion of the basin are operated by public and private entities whose operation provides for some protection of the public's water resource interests.

In the Canadian portion of the basin, the largest operator of dam and reservoir projects is British Columbia Hydro and Power Authority, but there are reservoir projects operated by others even there. Because of this, there are numerous committees, groups and organizations involved in coordinating the management of the Columbia River. Optimum regulation to one special interest may not be optimum to another and it is often difficult to reduce conflicts to a common economic denominator. Striving for optimum multiple purpose regulations of the Columbia River occasionally results in some controversy and conflict that may be resolved by Congress, in the courts, or in the public arena; but by and large, these conflicts are resolved through interagency cooperation.

The legal basis for coordinated operation of most of the hydropower generating facilities in the Columbia Basin is the "Pacific Northwest Coordination Agreement." This contract was signed by 16 parties controlling the major power facilities in the U.S. portion of the basin. Almost all of the reservoir storage within the basin is controlled either directly or indirectly by the signatories to this agreement. Reservoir storage is dedicated to coordinated hydropower use with certain limitations for non-power requirements. As a result of the Coordination Agreement it is possible for the owner of a downstream run-of-river or pondage project with no upstream storage under his control, to be assured of an amount of firm energy greatly in excess of what his project could produce without coordination. This concept of guaranteed firm energy is fundamental to the Coordination Agreement. In many respects this agreement provides an operating arrangement that approaches the optimum that would theoretically be possible under single ownership.

Each reservoir project is controlled within certain operating limits such as extreme maximum and minimum forebay elevations, rates of change in discharges and/or forebay elevations, minimum instantaneous and/or daily discharges, etc. In addition to these hydraulic limits, there are usually electrical limits imposed by project requirements or transmission system needs. Storage reservoirs in the Columbia system are regulated within these limits by a set of seasonal or annual reservoir elevation schedules or "rule curves." These rule curves are used to guide the operation of individual storage reservoirs as well as the entire system. The more important rule curves used to manage Columbia Basin storage reservoirs and the reservoir system, as illustrated in Figure 29, are:

-- "Critical Rule Curves (CRC)" are reservoir elevation schedules (Fig. 30) developed by annual operating studies using 40 years of historical streamflow records (1928-68) to determine optimum energy to meet firm loads during the most adverse water condition which may be as long as four years and is referred to as the "critical period."

-- "Refill Curves" are schedules of the lowest elevations to which a reservoir may be operated and still have an agreed upon probability of refill.

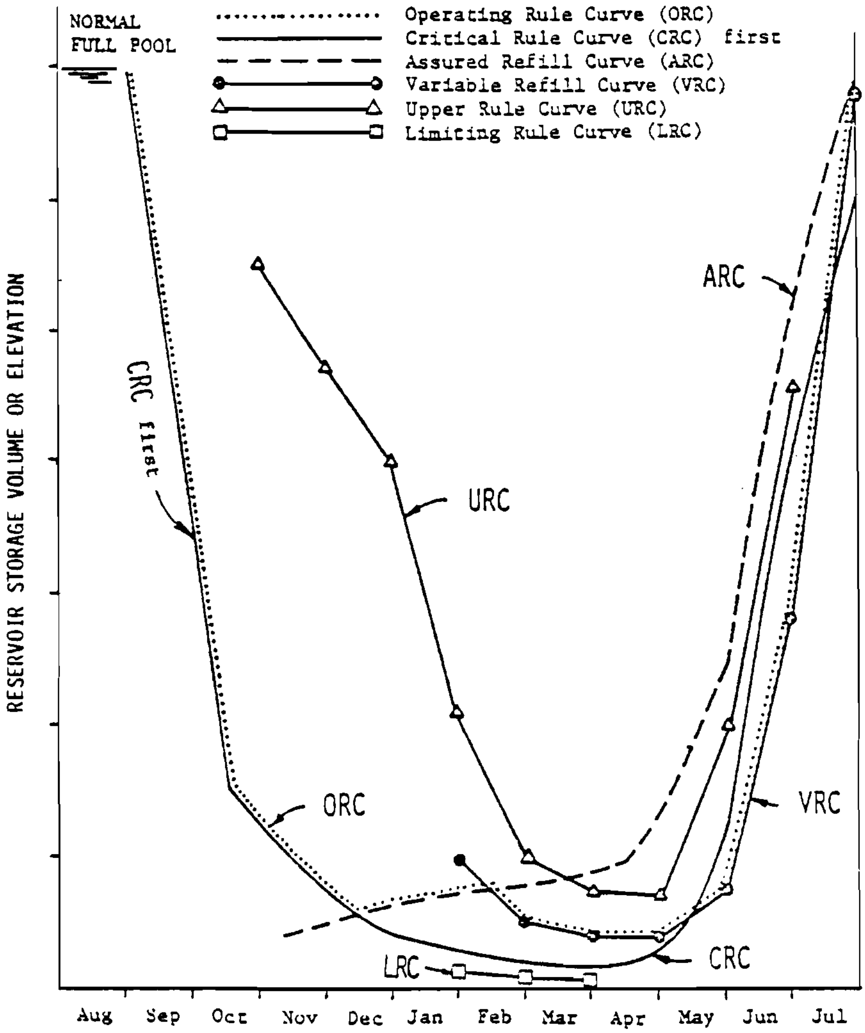


FIGURE 29. TYPES OF OPERATING RULE CURVES FOR A TYPICAL RESERVOIR IN THE COLUMBIA RIVER BASIN

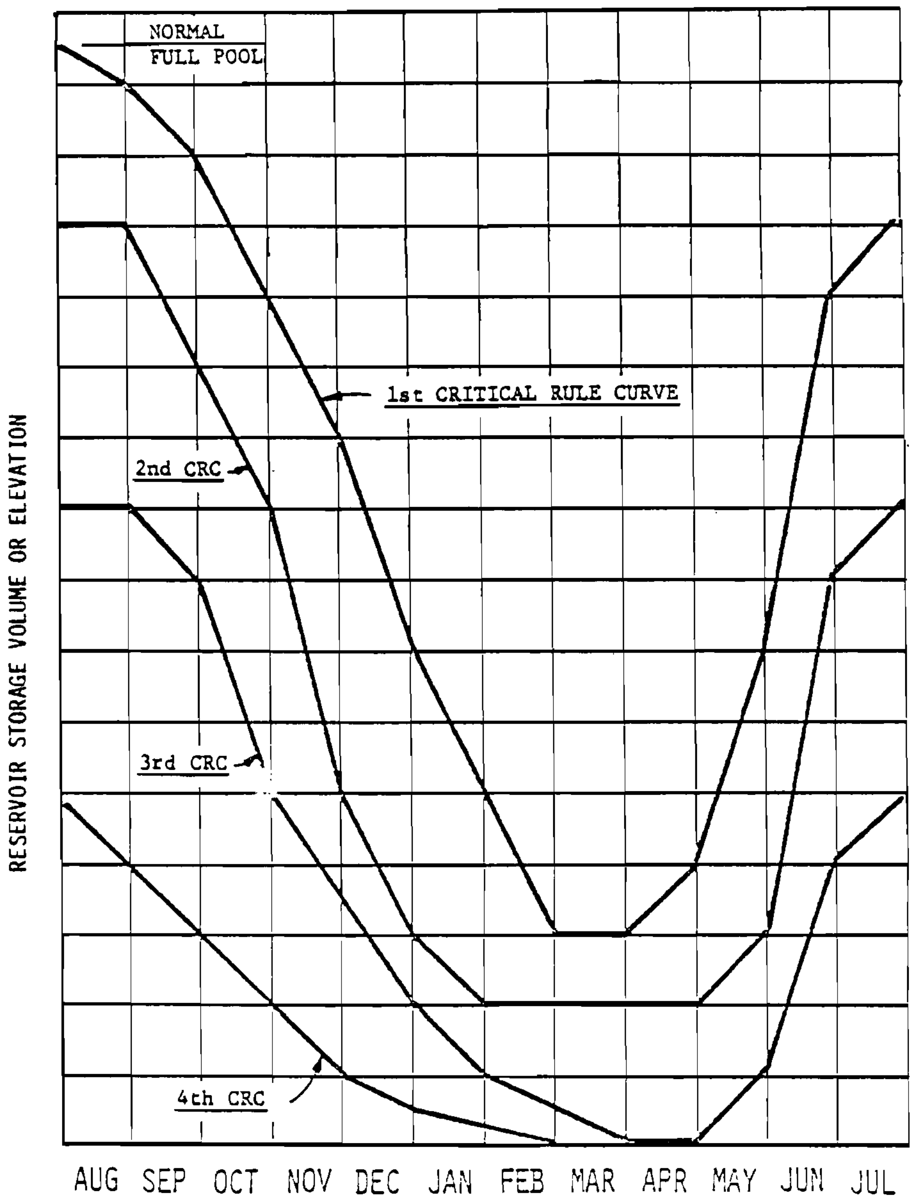


FIGURE 30. CRITICAL RULE CURVES FOR A RESERVOIR IN A MULTI-YEAR CRITICAL PERIOD IN THE COLUMBIA RIVER BASIN.

-- "Assured Refill Curve (ARC)" is a refill curve computed from the second lowest streamflow in the 40 years of historical record used for system studies.

-- "Variable Refill Curve (VRC)" is a reservoir refill schedule computed from forecasted volume inflows for the remainder of the current operating year ending 31 July. Water supply forecasts based on actual snow depth measurements and observed precipitation plus assumed subsequent precipitation are made periodically beginning 1 January each year for all major storage reservoirs and the basin as a whole. The inflow volume is usually reduced by deducting the 95 percent confidence forecast error and water required to meet loads during the refill period and to fill upstream reservoirs.

-- "Upper Rule Curve (URC)" is a reservoir elevation schedule indicating space required either during the evacuation or refill period to control potential flood flows.

-- "Limiting Rule Curve (LRC)" is a reservoir elevation schedule indicating minimum contents which must be maintained to guarantee the system meeting its firm loads during the January-April period in the event the variable refill curves permit storage to be emptied but sufficient natural flow may not be available until the start of the spring snowmelt freshet.

-- "Operating Rule Curve (ORC)" is the reservoir elevation schedule composed of segments from other rule curves as appropriate that will permit the maximum draft without jeopardizing system ability to carry firm loads in the future.

From the annual operating studies a family of rule curves are developed for optimum power production from individual storage reservoirs and for the combined system, as is illustrated in Figure 31.

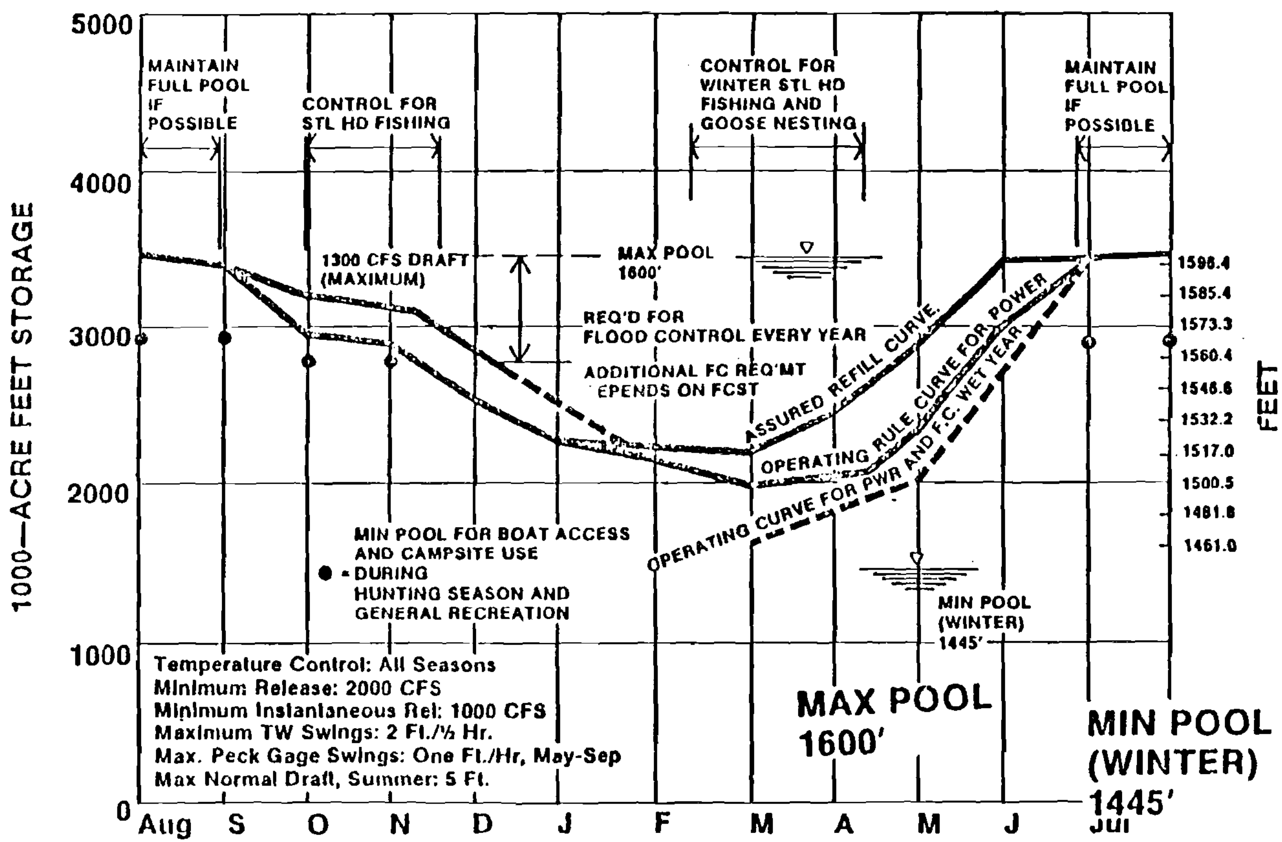


FIGURE 31. GENERALIZED MULTI-PURPOSE OPERATING RULES FOR A RESERVOIR IN THE COLUMBIA RIVER BASIN.

During actual day-to-day operation the reservoir owner/operators determine which rule curve is most appropriate under the conditions at that time. Some conditions change slowly such as streamflows during a period of recession, whereas some changes are rapid such as unscheduled outages of generating facilities. Non-power requirements can also force a change in power operations. These numerous variables must be handled for real-time reservoir management to be effective for hydropower purposes. Contingencies must be provided for and reserves made available. For many reasons the actual operations deviate from the optimum hydroelectric plan, while on the other hand efforts are being made continually to bring the system back into balance or to stay as close to optimum conditions as possible.

In daily operations, peaking capacity is usually of more concern than average energy requirements. Some projects are scheduled a day in advance, whereas others are used to make the instantaneous changes required to meet constantly changing load demands. Peaking plants are required to make rapid response to changing power demands and corresponding variation in discharges. Pondage projects are most commonly used to fill this need.

Computers are used in many roles throughout the Columbia reservoir system, from actual project control, to centralized system control, to system planning studies. Computer models are used frequently to make short range and longer range forecasts and simulations of the operating system. Project and hydrometeorological data are collected automatically, sometimes from remote sites, processed by computer, and then used for reservoir simulation studies. Human decisions and judgment have been the most efficient, effective and satisfying means of regulating the Columbia reservoir system when the persons in control are supplied with the best real-time information possible.

A simulation model developed by the Corps of Engineers is used 1) to help develop an operating strategy to be applied in current operations, 2) to assess proposed changes in operating strategies due to changes in both power and non-power requirements, and 3) to identify and assess future system additions or modes of operation.

In the past the Corps has developed and rejected two optimizing techniques for critical period operation for inclusion in the simulation model. These were iterative techniques that at that time were extremely costly in terms of computer time. However the Bonneville Power Administration and the Northwest Power Pool hydroelectric simulators have optimizers. While helpful for planning, however, they have not yet proved to be very useful or efficient for reservoir operation.

One reason for this is that economic measures of system performance for many of the multi-purpose reservoir uses are often difficult to determine. Examples are the possible extinction of fish runs, the unavailability of boat ramps for recreation use, or the lowering of reservoir levels below the inlets to irrigation pumps. It is even more difficult to quantify the wrath of the people who live around the reservoir that is being operated to meet such a requirement and can't. Hence it is unlikely that any optimization model will take the place of judgment during reservoir planning and operation, but of course such methods may be helpful in enhancing that judgment. So far this has not occurred with respect to reservoir operation in the Columbia River Basin.

CONCLUSION

This paper has been a review of multiple reservoir operation and planning and analysis methods used and proposed for use in North America. As is evident from the variety of these methods, this state-of-the-art ranges from rather simple policies and methods of analysis to rather complex and detailed ones. The seven actual multipurpose multiple-reservoir operating policies described in this paper illustrate this range of practice - from simple rule curves denoting ideal storage volumes for each reservoir and any necessary deviations from these based on judgment and experience, to more comprehensive computer programs used to determine detailed multiple-reservoir releases on a real-time basis.

Experiences of agencies responsible for the development and use of optimization and simulation models for improving multiple-purpose, multiple-reservoir operations in various river basins of North America are remarkably similar. At the risk of not citing all the exceptions, it is possible to summarize the typical problems encountered and the benefits obtained from such efforts.

1. Simulation models for daily, weekly and monthly operation have been found to be of value for aiding in assessing possible impacts of alternative operating policies and for forecasting the future state of the system given a specific operating policy and predicted hydrologic conditions.

2. The shorter the time interval, the longer and more costly the computer simulation. Because of the cost of computing it is often necessary in planning studies to use only two or three hydrologic years instead of the entire historical record, or sets of synthetic flow records, for a more thorough analysis of reservoir operation.

3. Optimization models have seen very limited application for multiple-reservoir operation. Difficulties include model development and cost of solution, the adequate inclusion of uncertain future hydrologic conditions, inability to identify and quantify all relevant objectives, and the need for better interaction with the user. Nevertheless, there is the general feeling that optimization techniques are potentially useful and their development continues.

4. The development of new quantitative analysis methods often takes many years. The communication problems that occur between research analysts and the practitioners in developing and applying these methods are real, time-consuming, and affected by institutional barriers. As Boston (9) has written, one can easily hear a practitioner saying:

> "We have met with these analysts each week for the past few months and they still do not understand our operational problems. They are more interested in their methods than in solving real problems."
>
> "Model development takes so long that the results won't be valid when and if they are ever available."
>
> "It takes a mountain of data to run the daily operating model and by the time the model is run, the forecast changes."
>
> "The models are inflexible. I have an optimal schedule now and need to know what to do when conditions are other than optimal."
>
> "Adapting a purchased software package to a real-world problem is like having a screwdriver and searching for a screw that it will turn."

Just as clearly, one can hear analysts saying:

> "Their operation is inefficient."
>
> "They do not want to understand what we are trying to develop."
>
> "They keep changing their needs - will they ever be able to decide what they want."

"Our problem is computer costs. What we need is a larger and faster computer."

"They wanted yesterday solutions to problems they gave us today."

These and similar comments that all of us have heard, and many of us have said, should not hide the fact that both simulation and optimization models have been of value to those responsible for the planning and operation of multiple-reservoir systems. Furthermore, their potential benefit is even greater. But before this benefit can be realized there is still considerable need for further research into methods that can define improved multi-purpose multiple-reservoir operating policies. These policies must be readily adaptable to changing hydrologic, economic and social conditions, and must be based on more accurate predictions of flood or drought conditions than are available today. Fulfilling these needs is indeed a challenge to those of us involved in the development and application of tools for defining improved multi-purpose, multiple-reservoir operating policies.

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METHODS FOR CONTROL OF COMPLEX RESERVOIR CASCADES AND THEIR APPLICATION IN THE USSR

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INTRODUCTION

On many large rivers in the USSR, cascades and hydropower projects have been constructed, which make seasonal, within-the-year and carry-over redistribution of streamflow (with respect mostly to time but sometimes to space) serve the interests of different branches of the national economy, e.g., agriculture, industry, water transportation, etc.

The high growth rate in water demand, particularly in the south of the country, as well as the goal of environmental protection, call for further intensification of hydraulic construction work. As a result, new water projects and more water resource systems are established and developed, and problems of unusual magnitude such as the transfer of water from the northern rivers to the southern regions of the country are considered. The hydropower plants in the USSR are complex, multipurpose projects, which constitute big energy systems, covering large areas. Furthermore, they are stochastic systems, in a very broad sense. The stochastic nature of these systems stems from:

1. The stochastic character of the main resources (streamflow occurrence, as well as measurement errors);
2. The stochastic and therefore uncertain pattern of water use and water losses;
3. Uncertainty in some of the economic data.

The above features are inherent in such systems, and they should always be taken into account when building mathematical models and applying optimization techniques for solving problems which arise during R&D and system operation. The stochastic nature of such systems to a large extent predefines the choice of control methods. The control methods in turn, depend on different characteristics of the reservoir complex, as well as on the operation of the energy and water systems served by the reservoir.

A few ways of classifying the operational conditions of such systems, together with their related reservoirs are described below:

1. Classification with respect to the extent of streamflow regulation by the reservoirs. A low level of such control is called passive control (Aturin and Reznikovsky 1976), i.e., when the reservoirs of the complex have practically no storage capability. In water and energy systems, the following measures may be considered as examples of passive control:
   a. Forecasting the system's operation under different water availability conditions, and for different levels of development.
   b. Defining the policy of setting the limits on and priorities of the use of scarce water resources, in conflict situations. This type of control is quite broadly used in water management practice in different countries. However, it cannot satisfy a stable, a fortiori, growing water demand.

Active control may be short-range (daily, weekly, monthly), seasonal and long-range (annual). The upper limit of such streamflow control is full control on an annual basis, when reservoir yield is close to the mean annual river flow.

2. Classification on the basis of disbalance in the system, which may be excessive, balanced, or deficient.
3. Classification with respect to the level of development. Depending on the level of development, reservoir control problems may be problems of design or else of operational control. In the former case, the systems' parameters, including the reservoir capacities and their operating rules have to be defined. In the latter instance, when the system already exists, we have a water management problem of an operational character. If design and implementation are correct, the reservoir operating rules stipulated in the project design are valid during a certain period of time.
4. Classification according to the purposes the reservoir is to serve. The reasons for constructing a reservoir may include (Kritcky and Menkel 1952): (a) providing (with a very high degree of reliability) the physical security of downstream constructions, and flood control; (b) providing for a certain guaranteed minimum water and energy supply with the reliability required by different water users; (c) providing for the maximum use of streamflow.

   In complex water systems, these three purposes are usually combined, though to some extent, they conflict with each other. For conflict resolution, it is necessary to use multi-criterion objectives or else to use a priority scale.

   In the USSR, safety features and flood control usually dominate all other objectives (Kritcky and Menkel 1952), while provision of an ensured, minimum supply (firm yield) takes priority over the maximum use of streamflow.

> Similarily, with a single objective (depending on the characteristics of water use or consumption in a region) it usually turns out to be possible to identify the sequence in which the demands of different water users are met in order to set the limits on supply when water is scarce, or else to find a policy for the use of surplus water (secondary yields).

In the USSR, various computational techniques, methods and principles are used (Kritcky and Menkel 1952; Tsvetkov 1967; Reznikovsky and Rubinstein 1974; Aturin and Reznikovsky 1976) for developing reservoir operating rules, depending on the conditions and characteristics listed above. The choice of these, as well as the necessary initial information, in particular the hydrological data, is to a large extent defined by the problem to be solved, and by its place in the above classification. For example, apart from an unconditional distribution curve for short-range passive control of streamflow, short-range streamflow forecasts (which are usually sufficiently reliable) are widely used at both the design and operation stages. For seasonal streamflow regulation, at both design and operation stages, long-range streamflow forecasts as well as field observation data in terms of multivariate probability distribution functions are used. For carry-over streamflow regulation, the most important information would be, at both stages, the probability distribution functions.

The hydrological data averaging intervals are also different. If, for short-range streamflow regulation as well as for flood control, hourly, daily, and less often, weekly intervals are used, then for seasonal regulation, weekly and monthly intervals may be used. For long-range streamflow control problems, the length of the interval may even be a year.

It should also be noted that the more deficient in water the system is, and the more complex and detailed the control measures, the more precise and reliable the methods for developing operation rules should be. For better results in the design and function of complex water systems, the development of mathematical models of water systems control now becomes very important. During actual operation, such models help to obtain definite recommendations on operating rule curves, while during the design stage of a project, models are used for computer simulation of the future operation of a system, which then leads to better design decisions.

Due to the fact that in many regions of the globe, including the USSR, water management problems have become more and more acute, more refined models, as well as newer reservoir control methods are being developed.

The need to protect the water resources of large river basins and the complex use of such resources generate a variety of problems, which in turn necessitates processing of enormous amounts of information. To cope with this situation, a new approach to the problems of planning and control of water systems (WS) of large river basins is needed.

As a methodological foundation for studying the functioning and development of WS in large river basins, a systems analysis principle is used, which includes the development of an interlinked system of mathematical models, and which makes possible numerous calculations for different environmental conditions, while taking into account individual features of the WS. The peculiar features of WS in large river basins create considerable difficulties in modeling, which have recently led to the use of simulation models of WS.

A simulation model of a large WS with information about the sites and features of water sources, all water demands, and sites and characteristics of all water projects as input, produces as its output operating patterns for all water projects and the system as a whole. Thus, the essence of using a simulation model of a WS consists in substituting a field experiment by a computer run, where the functioning of a real WS is represented by a computer simulation. Such a simulation system also serves as a tool for controlling the WS of a river basin, for it allows analysis of the consequences of implementing different alternative decisions.

Let us consider a rather general mathematical model of temporal and spatial streamflow redistribution, in a main fluvial river basin network, including reservoir cascades. In this model water movement in the fluvial network is described, and streamflow redistribution aimed at meeting various, and often conflicting, user demands (subject to certain optimality criteria) is sought. This model is the heart of a general system of models describing the operation and control of a water system, and models describing different branches of the national economy, as well as models of the sea are linked to it.

Although a number of computerized models for optimizing the operation of complex cascades of hydropower stations have been developed in the USSR, the ones most widely used in every day practice are those marked with "PK" (cascade calculations) (Tsvetkov 1967; Kuzmin et al. 1977). The "PK" codes may be applied to any cascade of hydropower plants and even to the modeling of groups of cascades (up to 20 jointly functioning hydropower stations located in one or many channels). These codes are oriented toward a deterministic input data concerning streamflow, water demands, etc. Successive corrections are used to update the forecasts so as to fit the observations. In the "PK" codes, a regu ation period--a year or more--is subdivided into discrete intervals of 10 days, a month, or a few months. The water balance equations have the form:

$$W_{ij} = W_{(i+1)j} + (Q_{pij} - Q_{r\partial cij} - Q_{xcij} - Q_{uenij} - Q_{\omega ij} - Q_{\varphi ij} - Q_{\delta\sigma ij}) + \sum_{\eta}[(1 - \frac{T_{i\eta}}{\Delta_{ti}})Q_{\mu\sigma i\eta} + \frac{T_{(i-1)\eta}}{\Delta_{ti}} Q_{\mu\sigma(i-1)\eta}]\Delta t_i \quad ,$$

where

Q_{pij} - is the total inflow to j-th hydropower stations (HPS) reservoir during i-th time interval;

$Q_{r\partial cij}$, Q_{xcij}, Q_{uenij}, $Q_{\omega ij}$, $Q_{\varphi ij}$, $Q_{\delta\sigma ij}$ - are water discharges through HPS, the dam, evaporation, navigation locking, sewage, and use, respectively;

η - are the numbers of the upstream HPS adjoining the j-th one;

T_{ij} - transition time for j-th and (j+1)-th cascade stages;

W_{ij}, $W_{(i+1)j}$ - j-th reservoir capacity at the beginning and the end of i-th time interval respectively,

i - the time interval.

Despite the fact that the code deals with static capacities, the introduction of transition times (Tsvetkov 1967) describes as a rough approximation the reservoir capacity dynamics. The optimality criteria in this code are formulated in terms of energy production--maximization of HPS power output. In addition, more complex criteria, dealing with all power plants of the energy system and oriented toward minimization of the total fuel consumption by the thermal power plants are used.

The demands of non-energy water users and consumers are reflected in the constraints, where the lowest admissible water level along the entire length of the river, and water releases necessary for fish-breeding and irrigation, as well as other constraints, are explicitly described. The constraints also include some energy-related operational conditions, e.g., peak and intermediate power output of hydropower stations, capacity of electricity transmission lines, etc.

For finding the optimal solution, the projected gradient method is used in these codes (Kuzmin et al. 1977).

The "PK" codes have been used in the USSR for a number of years in daily practice, for control of large HPS cascades. Using these routines at the design stage for deterministic calculations with a hydrograph series, one can obtain regular dispatcher operating rule curves, e.g., with the help of regression analysis. However, in design practice in the USSR, other dispatcher rules have become much more widely used. These rules are now referred to as heuristic, and their detailed description is given in Kritcky and Menkel (1952), and Reznikovsky and Rubinstein (1974).

Simulation of a system's behavior under different hydrological conditions, and development of the heuristic rules for varying parameters of the system are usually computerized. So even in the early stages of system design it is possible to take future working conditions, such as dynamic reservoir capacities, sanitary constraints, etc., into account.

It is appropriate to give here a short review of the existing cascade control methods, using as an example some reservoir cascades which exist or are under construction in the USSR.

THE ANGARA-ENISEI CASCADE

The Enisei is the biggest river in the USSR. Its basin area approaches 2.5 million km^2, while mean annual streamflow is 585 km^3. The total length of the river basin is almost 90,000 km. The theoretically possible hydroelectric potential of this river is close to 600 billion kwh/year (Voznesensky 1967). Six hydroelectric plants, with a total installed capacity of over 10 million kw and a mean annual output of about 70 billion kwh (Dimitrevsky 1962), are scheduled to be built on the Angara river.

A characteristic feature of this cascade is the very high degree of streamflow regulation made possible by construction of the Bratsk reservoir, and damming of the Baikal lake.

The Angara's energy resources were tapped by building the hydroelectric plant with an installed capacity of 660 MW in the upper reach of the river. The next step was the construction of the Bratsk HPS with a capacity of 4,100 MW. Construction of the third stage of the cascade (Ust'-Ilim Power Plant) is now close to completion. In its first phase it will have the installed capacity of 3600 MW. The total annual energy output of these three Angara hydropower stations will be close to 50 billion kwh (Dimitrevsky 1962). The fourth stage of the cascade--Boguchanovskaya hydropower plant--is currently under construction. Its installed capacity will be 4000 MW, and mean annual output 17.8 billion kwh.

On the Enisei river, the Krasnoyarsk HPS has also been put into operation; its installed capacity is 6000 MW, and the mean annual output is 20 billion kwh. The Sayano-Sswshenskaya HPS, with a 6400 MW installed capacity and 23.5 billion mean annual output is being built (Voznesensky 1967). Enisei, Igar and other hydropower plants are in the design stage. Research on the possibility of building large power plants on lower reach tributaries of the Enisei is going on. Research on the joint operation of the Angara and Enisei hydropower plants in the Siberian energy system is of particular interest.

Working from the fact that tne natural, unregulated streamflows of the Angara and Enisei are asynchronous and that these are differences in the regulating capabilities of their reservoirs, special rules for operation of these power plants were developed. These rules were called "Interbasin Electricity Compensation Regulation";they are used for regulation of the hydropower plants' energy output. In accordance with the rules, the upstream Enisei HPS which had rather weak reservoir regulating capabilities was assigned to a lower level in the control hierarchy, and it was allowed to operate so as to optimize use of the Enisei streamflow. The Angara HPS constituted the upper level in the control hierarchy. These two play a compensating role by raising the fluctuating output of the Enisei hydropower stations up to the value guaranteed to the system with a certain reliability (Reznikovsky 1969). As a result, the overall guaranteed output of the Angara-Enisei cascade was raised considerably. For example, the increase in the guaranteed output of only three HPS of the cascade exceeds 500 MW. This caused a decrease in the capacities of the thermal power plants by this value, which saved tens of millions of rubles. When eight hydropower plants are in operation, this value will increase by a factor of three (Reznikovsky 1969). One should not forget, however, that water resources of the Angara and Enisei are used not only for electricity production, but for water transport, water supply, and recently also for recreation. The reservoir operating rule curves for this cascade take into account the need to satisfy the demands of these users with a given reliability. Thus, even at the design stage of the Angara-Enisei hydropower plants and reservoirs, the systems approach was widely used. This helped to deal with the problem by taking into consideration a variety of environmental, technical, and economic factors which influenced the effectiveness of HPS operation in the complex water and energy system of Siberia.

The use of the systems approach in operating the hydropower plants in this region raised their economic effectiveness. Together with traditional methods, new techniques developed recently were employed. The methods of statistical modeling of streamflow, developed specially for the Angara-Enisei cascade are described in the literature (Reznikovsky 1969, and Reznikovsky and Rubinstein 1974), and are now in constant use in designing other cascades. Techniques were developed for long-range operating control of hydropower stations that have been used for a number of years. These techniques allow implementation of optimal operating rules defined at the design stage. It should be noted that the optimality criteria are rather sophisticated. For optimization of a multi-dimensional objective function, different iterative methods are used, including gradient methods, dynamic programming approach, etc. For deducing regular operating rules from optimal solutions, various heuristic and regression methods are applied.

In order to reduce the dimensionality of the optimal control problems (initially for the Angara-Enisei cascade of HPS) the principles and practical techniques of hierarchical control were developed (Reznikovsky and Rubinstein 1974). The introduction of the hierarchical structure makes it possible to decentralize to a large extent, the problem of operational HPS control for some hydropower plants. It makes these problems partly autonomous, without practically any loss of benefit in joint HPS operation (Reznikovsky 1969, and Reznikovsky and Rubinstein 1974).

Recently, for operational control of the Angara-Enisei hydropower plants, an attempt to use a new branch of probability theory--the theory of controllable stochastic processes--was made. In such an approach, the main relation is a stochastic differential equation of water balance. The streamflow is approximated by a continuous harmonizable Markov process. The analytical description of streamflow is given by an autoregression equation in transformations of streamflow values, which fit with the Gaussian distribution. The constraints on control are treated by penalty function, and the type of functions depend on the availability level of the constraints.

The system of stochastic differential equations describing the Angara-Enisei HPS cascade, the objective function and control have been linearized piece-wise. This has allowed a reduction of the optimal control problem to the solution of the matrix Riccati equation of a small dimension (Reznikovsky and Rubinstein 1974).

THE VOLGA-KAMA CASCADE

This cascade includes 9 hydropower plants, with two more under construction. The cascade of reservoirs provides within-the-year streamflow regulation. The main water users in the Volga basin are the hydropower plants, water transport, fishery, irrigation, and industries. The hydropower plants have important functions: they are the source of electrical power and cover peak-load demand, they regulate the frequency in the energy system and provide the maneuverable blackout reserve. Construction of the reservoirs on the Volga river, Volga-Don Canal as well as the reconstruction of the Volga-Baltic waterway have turned the Volga into a deep-water transport route which is constantly in use. The reservoir construction program provided for intensification of the irrigated agriculture in the valley. Fishery is also a very important water user in the system; 50% of the country's total fish yield from inland waters and 90% of sturgeon yield is from the Volga-Caspian basin.

The demand structure in this basin is highly time-dependent. For this reason, it is very convenient to break a year into three parts: spring (flood-time), navigation (vegetation season), and winter (normal water season). During the flood time, releases into the lower course of the river, aimed to meet the demands of agriculture and fish-breeding, have the greatest impact on the operating regime. These releases may happen to by-pass the hydropower plants and may result in non-filling of the reservoirs by the end of the flood-time, which in turn leads to high economic losses. During the navigation period, operating rules are mainly defined by the irrigation and transport needs. In winter, the main target becomes energy production and meeting peak electricity demand.

A guaranteed supply to the energy producing branches of the national economy are considered as given when analyzing the operating conditions. They are defined as a result of a special analysis, and remain constant for a number of years. The spring fish breeding release is defined anew each spring, usually on the basis of expert recommendations which take into account the specific climatic conditions during that year (rainfall-runoff forecasts, energy demands, etc.). Provided the uniform demand of non-energy-producing branches of the national economy is given, the optimal operating rule curves for the cascade of reservoirs are found with the use of the above mentioned "PK" codes. Due to the probabilistic nature of the forecasts, successive corrections are made during the year.

In calculations for the nearest period of time, a streamflow forecast is used, while for the rest of the year, the information is limited to the conditional expectations of streamflow.

To meet the guaranteed demand of different water users, the "PK" codes make separate calculations for low water hydrographs of a calculated recurrence. These calculations permit definition (as the upper envelope) of the curve of the minimum reservoir water level, which cannot be decreased, so as not to fail to meet the guaranteed demand in the next period.

In practice, the operating service makes dozens of calculations during the year, using the "PK" codes, to keep system operation at the optimal level. One of the most complex problems of cascade control is the distribution of scarce water resources between users, in a conflict situation. This problem may be solved in two ways:

- -- on the basis of loss functions so as to minimize the sum of losses;
- -- on the basis of given priorities.

In the former case, the most difficult part of the problem is to define the loss functions for different branches of the national economy. For the energy producing branches, the loss function is defined with the use of the routine "PK": this code enables calculation of the reduction of HPS output, in relation to guaranteed output, and then with the specific losses per unit kwh known, the total losses may be defined. The losses in other branches of the national economy are defined by respective experts. The clear drawback of such an approach is that the loss functions so defined are rather rough approximations of reality.

The problem of water distribution in a conflict situation is now quite often treated on the basis of preset priorities. In such an approach, mathematical models of reservoir control are used for simulation purposes: they help identify operating rules for different priorities, when there are limitations on supply. A successful analysis allows the decision maker to refine the priorities.

NARYN-SYRDARIA CASCADE

The water system built within the limits of the Syrdaria basin now covers an area of 0.45 million km^2 and ensures half the industrial and agricultural output of the mid-Asian region of the USSR.

The infrastructure and development of the WS of the Syrdaria basin is predefined mainly by the needs of the single major component of the system, namely, by irrigated agriculture. For securing a stable water supply, as well as for melioration of irrigated land, about a million irrigation canals were built, with a total length of over 50,000 km. Together with the irrigation canals, the water system includes a dense network of drainage canals with a total length of over 30,000 km. To improve the water supply to the irrigation system in the flat areas of the basin, nine water-level-raising dams were built. The need to satisfy the growing demand during dry seasons and years has predefined the necessity to build reservoirs for regulating streamflow of the river and its tributaries. The effective capacity of eight existing reservoirs is 10.3 km^3. Four additional reservoirs with a total capacity of 23.1 km^3 are under construction. The biggest among the existing and planned reservoirs are the Toktogul, Charvak, Kairekkum and Chardaria reservoirs. In addition to agriculture, hydropower generation is an important factor in the basin.

The general formulation of the control problem for the Syrdaria system is as follows: find the operating rules for all the water projects of the system, with the use of the information on:

- sites and peculiarities of the available water resources (including return water) in the regulation zone;
- operating rules and water demands of aggregated water users;
- sites and working characteristics of all water projects.

A simulation of the system should lay the foundation for solving the following water management problems:

- definition of reasonable limits to which water resources of the basin can be used, and consequently, the reliability of water supply for separate and aggregated users and the system as a whole;
- definition of the conditions concerning external water resource needs at different stages of the system's development;

-- choice of rational operation regimes for the reservoir system (both with and without interbasin links), and to estimate the possible influence of these regimes on the projects and environmental conditions in the adjoining territories;
-- development of the operating rules for the reservoir cascade, as well as for other system projects.

The planning of the development of the basin's water resources is now based on the principle "bottom first." A plan is worked out by analysis of individual water users and consumers, whose demands are then summed up within the administrative regions. Using this information, forecasts, and available data concerning streamflow and characteristics of reservoir site conditions, the necessary calculations are then carried out. If users' demands can be met, the reservoir operating rules are then defined by dispatcher rules.

All operational alternatives for the Syrdaria system reservoirs are checked through the water management calculations. The choice of the final variant is made by experts. If water is scarce, the planned supply is cut down so as to balance the real supply and demand. The percentage reliability of water supply is defined by experts on the basis of established standards and information supplied by special committees especially established for the fair distribution of water.

In high water years, if the danger of land flooding is acute, decisions about forcing the reservoir levels, special releases, spillover and other flood control measures are made. Plans are usually made for long periods (vegetative and winter, subdivided into months), but also for periods from a day to a month.

Development of irrigation in the basin and construction of new reservoirs with carry-over regulating capabilities and powerful hydropower stations creates new demands on the methodology of defining optimal operating rules for the system. This also forces a lengthening of the planning horizon and makes the management problems much more complicated.

The simulation model for the Syrdaria basin, which is being developed now, does have to help solve water management problems in the basin under constantly varying conditions.

The following principles were chosen as a basis for the system of simulation models.

1. Water availability for the users in low-water periods may be raised by presetting the minimum level of water supply.
2. The cut in water supply is forecasted for each user in accordance with a special table. This table contains information about the feasible limits of water supply.
3. In limiting the demands, users' priorities are taken into account, which are presented in a table where the users are listed according to their importance.
4. The excess water is balanced out among the reservoirs, first by the nearest one, and then one by one, downstream. As soon as the highest permissible level is attained, the remaining water is discharged into the Aral sea.
5. In high-water years, in case the danger of flooding is present, special decisions about forcing the reservoir levels, special releases and additional spillover capacities are made.
6. The deficit, if any, is met by releases from the nearest upstream reservoir till its water level attains the minimum limit. Next, water is released from the upstream reservoirs. If these measures fail, the supply is reduced in accordance with the users' priorities.
7. Filling all the reservoirs at the end of the last period of a year should correspond to a preset level depending on the climatic conditions prevailing in a given year. The upstream reservoirs take priority over the others. If the preset levels are unattainable, the correction of their final levels is made with respect to their priorities and the user's priority table.

Experimental runs of this simulation model have allowed estimates to be made of the reliability of water availability for separate and aggregated water users as well as for the basin system as a whole. They helped also to evaluate the extent of the deficit under different climatic conditions, and for different stages of the system's development.

THE DON WATER SYSTEM

The Don water system consists of complex waterways and water projects supplying water to various users, mainly for irrigated agriculture and pasture inundation, water transport, industrial and urban use, fishery, energy production and the maintenance of sanitary flows in the lower course of the Don.

The main water projects in the system are the Tsymlyanskoe reservoir, for carry-over streamflow regulation, and Nikolaev and Kochetov low-head dams, which maintain sufficient levels for navigation on the lower Don. Besides that, construction of the Konstantinov low-head dam has begun, and building of the Bogachev regulator is planned. These projects will complete the measures taken to improve the transport conditions in the lower Don.

The current water situation in the Don basin is defined mainly by the growth of water demand at the upper and the lower course of the Don, as well as by the need to release substantial amounts of spring water from the Tsymlianskoye reservoir. This considerably tightens the operating conditions in the system and requires qualitative changes in the operating rules developed earlier. A system of mathematical models has been developed to investigate the Tsymlianskoye reservoir control. This allows simulation of the long-term operation of the WS and analysis of different operating rules under various climatic conditions.

This methodology was used both for analysis of the traditionally adopted dispatcher rules and for development of special operating rules for spring fishery passes from the Tsymlianskoe reservoir using the hydrological forecast of the spring inflow capacity, made 2-5 months beforehand. The results obtained were analyzed from the point of view of the USSR indexes of water supply reliability concerning different water use categories, and judged by the efficiency criteria of the runoff use. These results allowed decisions to be made both in an expert way and by using loss functions derived on the basis of different hypotheses against the shortage of water in different components of the WS. The simulation model of the Don WS was used for more accurate definition of operating rules of the Tsymlianskoye reservoir. Besides that, the model made possible a large range of investigations on different water systems' operational aspects, such as:

1. Questions of reliability of water supply to different water users under various operating rules.
2. Dispatcher's operating analysis, methodology of preparing the dispatcher rules; impact of the location of controller's lines on the level and time of water shortage; calculation of the impact of water supply regularity on the dispatcher rules.
3. The analysis of permissible relationships between the guaranteed water supply provided and the degree of runoff use under the conditions of complex water resources use.
4. Establishment of the initial data for comparative analysis of such methods of management as dispatcher rules, successive corrections, and table of priority.
5. Defining the permissible degree of specifying concretely the operating rules when solving the tasks of WS at different levels.

Analysis of the above-mentioned questions and the improvement of simulation models are subject to further investigation.

CONCLUSIONS

The development of reservoir cascades is characterized now by the complexity of relationships between a water resources system, its environment and various branches of the economy, by the contradictory character of different water users' interests, and by the difficulty encountered in expressing these interests in terms of one criterion.

The problems concerning operation of such reservoir cascades can be solved to a large extent by using the system of simulation models which will permit evaluation of the reaction of the water system to the change of its input characteristics. In this way, a vast amount of information about the way the system functions under different conditions can be derived for analysis and decision-making.

The impact of the use of the simulation models to a great extent depends on the available information on the size and characteristics of the cascade, about water resources, water use and economic information defining the rules of cascade operation. Accuracy and the type of information depend on such factors as the stage of the operational task decision, depth of runoff regulation, structure of the water system, etc.

Water systems belong to the class of large systems with partly undefined and probabilistic information. Specifying the size and operational regime of such systems relates to the class of two-step stochastic optimization tasks. Taking into account the multipurpose character of water systems, the vector-function stands for an optimal criterion. Examples of the use of simulation models presented in this paper illustrate the application of the systems approach to water management problems.

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REAL-TIME AND LONG-TERM ASPECTS OF OPERATING MULTIPURPOSE RESERVOIRS

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1. MULTIPURPOSE RESERVOIRS

Most reservoirs, in particular the large ones, have multipurpose functions. The various uses of reservoirs may be grouped under the following categories:

a. Water supply: - municipal (drinking water etc.)
   - agricultural (water for irrigation)
   - industrial (water for production, for cooling etc.)

b. Flood control: flood flow retention in order to prevent inundation

c. Energy production (hydroelectric power)

d. Flow augmentation, in particular during low flow periods, to guarantee in the downstream river section:
   - the required minimum flow,
   - sufficient water quality (considering the unavoidable wastewater releases into the river)

e. Recreation, fishing etc.

2. GENERAL PROBLEMS IN RESERVOIR OPERATION

According to the increasing water demand and to the growing amount of waste water, the reservoir functions mentioned above under (a) and (d) have become increasingly important. They are characterized by a typical seasonal variation with a remarkable demand peak in dry and hot summer periods when the natural water yield is generally low.

During such periods, the users try to satisfy their increased demand from the reservoirs. Therefore, the responsible water authorities are interested in storing as much water as possible during periods of increased flow, in particular during flood periods. This leads to the following principal problem in reservoir operation:

- the desire to reduce the flood control volume of the reservoir in favor of increased water storage for low flow periods (problem of reservoir space allocation).

The other general problem is:

- to find an optimum or at least reasonable strategy for the distribution of reservoir water among the different water uses to be supplied (allocation of reservoir releases).

3. MODEL SYSTEMS FOR REAL-TIME CONTROL AND LONG-TERM SIMULATION

The solution of both problems is more critical the higher the water demand is in comparison with the available water resources. Under such conditions it is necessary:

1. to design and install an efficient real-time forecasting and control system in the river basin which enables the responsible water authority to control the

reservoir releases of a predetermined "optimum" long-term control strategy, and

2. to simulate the "natural" flow regime of the river basin over long periods as accurately as possible and to derive a control strategy which provides an optimum long-term water supply for all important users.

In the GDR, for the solution of these tasks comprehensive systems of mathematical models and computer programs have been developed and applied. In keeping with (1) and (2) they are denoted as:

a. real-time forecasting and control models, and
b. long-term simulation models.

Both model systems have become important tools in the management of water resources systems (WRS), especially of those with reservoirs. Their basic structure is the same as represented schematically in Figure 1. Main differences are:

- the real-time model (1) is coupled with an operational data sampling and transmission system (see Figure 2) and the computation time increments are adequate to the processes (one or some hours)
- the long-term simulation model (2) requires greater computation time increments (usually one month) and accordingly simplified versions of some submodels of the WRS.

Further information is given in Becker (1977, 1978), and Lauterbach and Becker (1977).

4. OPTIMUM REAL-TIME RESERVOIR CONTROL

Main functions of the real-time control model are (with regard to reservoirs):

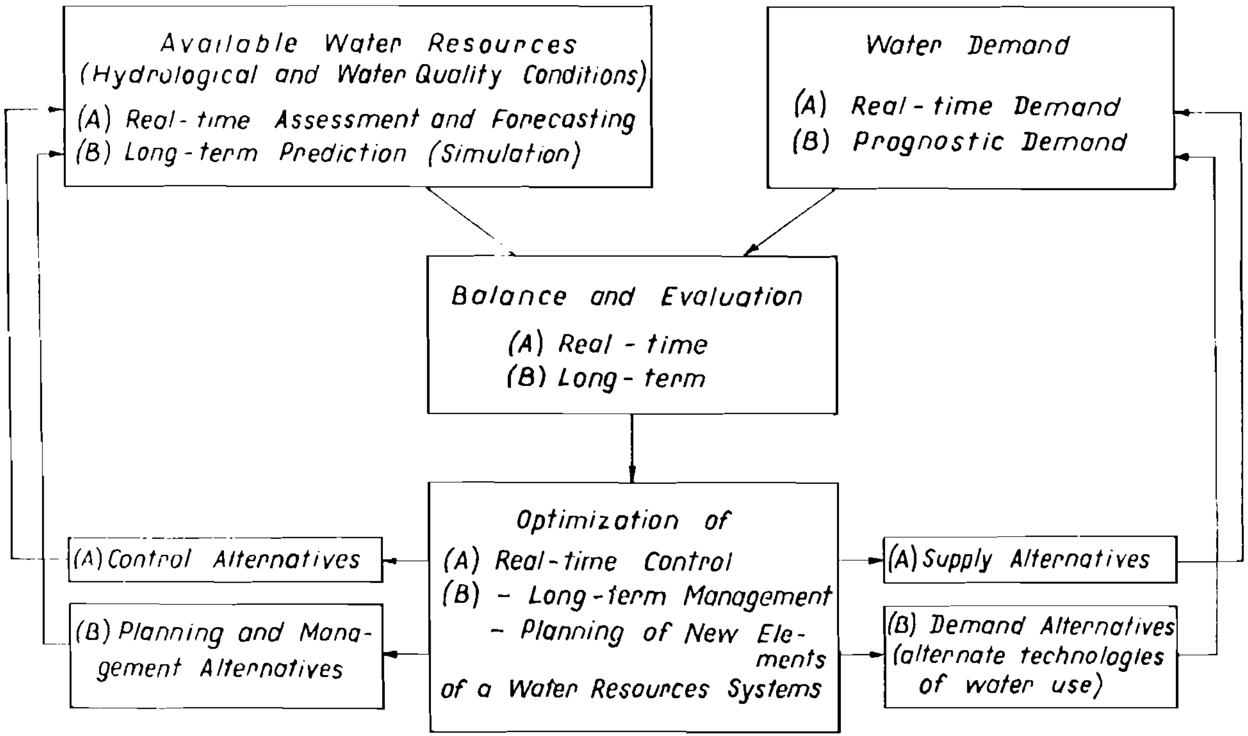

Figure 1. Main parts of a WRS model and decision alternatives with special regard to operational control (A) and long-term management (B).

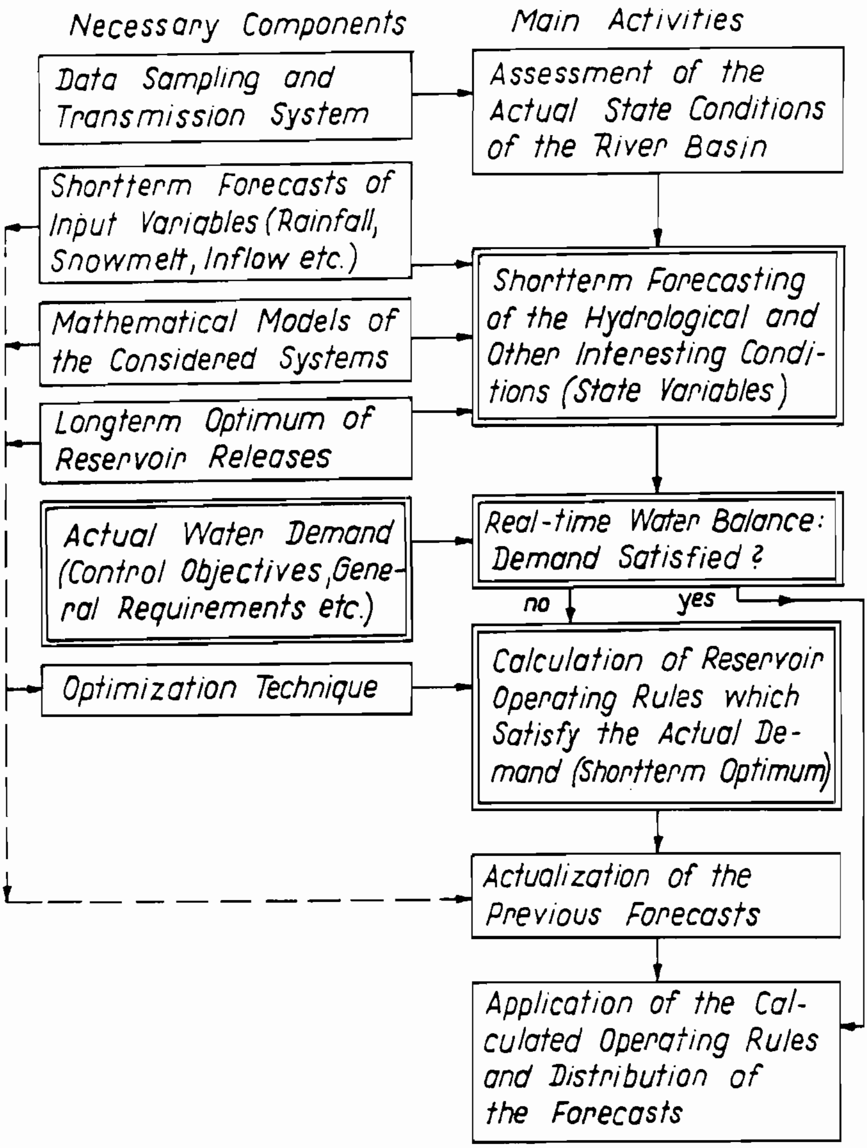

Figure 2. Necessary components of an integrated operational hydrological forecasting and control system of a river basin, and main activities in its application.

- short-term forecasting of the hydrological and other state variables in the river basins to be controlled (some days or weeks ahead)
- calculation of operational control rules for the reservoirs which guarantee an "optimum" satisfaction of the actual demand considering:
  - o the forecasted state conditions (feedforward control)
  - o the predetermined optimum long-term control strategy.

The necessary components of those models and the main activities in their application are presented in Figure 2 (as a refinement of part A of Figure 1). Further explanation is given in Becker, 1977.

In the following, only the most important control cases requiring rather detailed and accurate control calculations should be considered:

- flood control
- low flow control
- water quality control (waste concentration in rivers).

In all these cases the objective of the control calculation may be expressed as follows:

- Definite "requirements" (control objectives) $Z_K(t)$ are given for selected cross sections K downstream of the reservoir (mostly constant or seasonally varying maximum and/or minimum flows and/or maximum waste concentrations along the river).
- The state variables $Q_K(t)$ to be controlled (actual flow or concentration) should approximate these control objectives $Z_K(t)$, but should not exceed them.

Thus, the objective function for the control calculation may be written as:

$$\int_{t_0}^{t_1} (Z_K(t) - Q_K(t))^2 \, dt = \text{MIN} \tag{1}$$

where the time period t_0 until t_1 is the nearest controllable future (see Figure 3; feedforward control).

For simplicity, we omit the index K in the following, keeping in mind that the equations apply in the same form to all interesting reference cross sections K.

Q(t) consists of two parts:

- a noncontrollable part QN(t) (e.g., natural runoff of uncontrolled parts of the river basin)
- a controlled part QCO(t) related to the controllable release function u(t) of the reservoir (or of any other water management installation) via an operation F:

$$QCO(t) = F[u(t)], \qquad u \in U \tag{2}$$

U - set of feasible control functions. In the case of linear systems, F may be replaced by the convolution integral:

$$QCO(t) = \int_{t-t_N}^{t} u(T)h(t-T)dT = \int_{0}^{t_N} h(T)u(t-T)dT \tag{3}$$

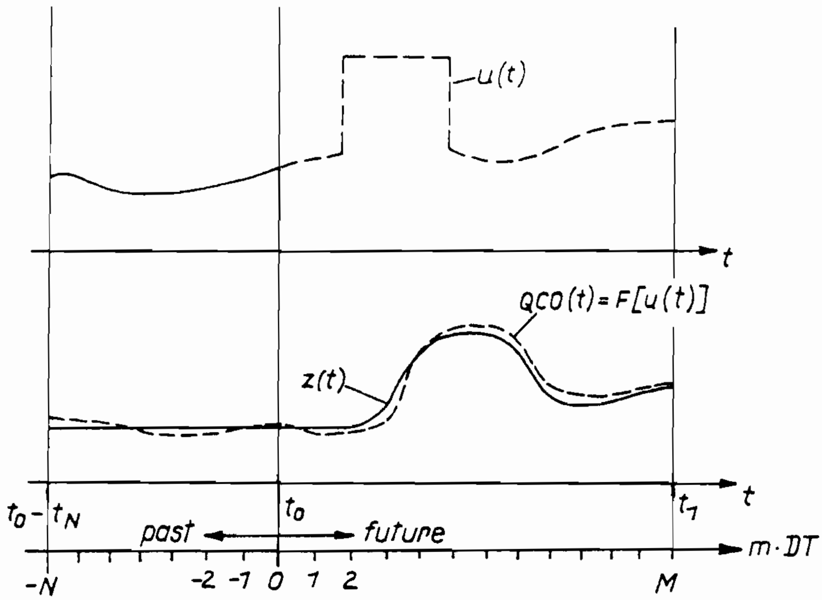

Figure 3. Definiti:n sketch for real time control calculations.

or, in discrete form:

$$QCO_m = \sum_{i=0}^{N} h_i \; u_{m-i} \qquad (4)$$

where h is the pulse response function of length t_N (or maximum number N of ordinates in distance DT). According to Q(t) = QN(t) + QCO(t) we can write:

$$Q_m = QN_m + \sum_{i=0}^{N} h_i \; u_{m-i} \qquad (5)$$

m = 1,2,3...,M (according to t_o until t_1; see Figure 3). Then the objective function $(1)^o$ may be written as:

$$\sum_{m=1}^{M} ((Z_m - QN_m) - \sum_{i=0}^{N} h_i \; u_{m-i})^2 = \underset{u_k}{MIN} \qquad (6)$$

The task is to find in each interesting situation t_0 such a function $u_k(k=1,...,M)$ which makes the expression in example (6) minimum (optimization as indicated in Figures 1 and 2). The ordinates u_k can be determined directly if a so-called "smoothness" constraint is introduced in example (1):

$$\int_{t_0}^{t_1} [\lambda \; u^2(t) + (Z(t) - Q(t))^2] \; dt = \underset{u}{MIN} \qquad (7)$$

where λ is a weighting parameter (the greater λ the smoother u). Then equation (6) reads as follows:

$$\lambda \sum_{m=1}^{M} (u_m - u_{m-1})^2 + \sum_{m=1}^{M} ((Z_m - QN_m) - \sum_{i=0}^{N} h_i u_{m-i})^2 = \underset{u_k}{MIN} \qquad (8)$$

The differentiation of this equation by $u_k (k=1,...,M)$ leads to a system of linear equations for the u_k.

Equation (6) clearly reveals two important facts:

- the optimum release function u depends on the non-controllable part QN of the system output
- under certain conditions the objective function Z cannot be fulfilled by controlling u (e.g., in the case of floods, if QN already exceeds the limit Z).

Both facts indicate the major importance of accurate short-term forecasts of QN.

5. APPLIED TECHNIQUES OF REAL-TIME CONTROL CALCULATIONS

Real-time control calculations for reservoirs have been carried out in the GDR for the following cases:

a. Flood control:

Calculation of the maximum acceptable reservoir releases u which guarantee that the inundating discharges Z along the downstream river section will be nearly reached but not exceeded (if the noncontrollable flow QN is smaller than Z). This calculation is required:

- to prevent inundation during the peak flow period,
- to release the storage of the flood control volume VF of the reservoir as soon as possible during the flow recession and,
- to get additional storage volume before the peak flow period.

b. Low flow augmentation:

Calculation of the reservoir release u necessary to guarantee the required minimum river flow (Z). Excesses that might be caused by short-term natural flow rises during low flow periods, should be avoided in order to save as much water as possible for other purposes and for the subsequent period.

c. Control of salt concentration within a given limit: Calculation of:

- the permissible maximum salt wastewater releases from smaller wastewater reservoirs which can be discharged by the actual river flow without exceeding the given salt concentration limit. It should be pointed out that a successful operation of a control system of this type is only possible, if the wastewater reservoir and the fresh water reservoir are in parallel so that mixing takes place at the confluence of the two rivers.
- releases from fresh water reservoirs necessary to prevent forecasted excesses of the given limit value.

In the case of linear systems these control calculations can be carried out on the basis of example (8). Another technique which can be applied also in the case of complex and non-linear systems is an alternative search for the optimum function u_k on the basis of equation (6). The main steps of this technique are (according to Figure 1, part A):

- estimation of the necessary amount of reservoir releases and of their travel time T = T(Q) to the interesting river cross section:

$$u_{m-IT} = Z_m - QN_m \tag{9}$$

where IT = T/DT = approximate number of time intervals DT.

- forecasting of the resulting Q(t) by means of available routing models
- iterative correction of the amount of the control releases u and of the time lag IT until the calculated system output Q(t) adequately fits the control objective Z(t).
- If different control sections K have to be considered, separate calculations can be run for each of them. Finally that release function u_K can be selected which fulfills all functions $Z_K(t)$.

This technique has successfully been applied within the operational system for forecasting and control of river flow and salt concentration in the central Saale river and for flood control in the Bode river (Becker, Sosnowski, 1977; Becker, Krippendorf, Thiele, 1978).

6. LONG-TERM MULTI-SITE FLOW SIMULATION

The required reliable long-term simulation of the flow regime in a river basin has been developed in the GDR on the basis of a stochastic simulation technique (Monte-Carlo technique) considering the flow process as a multi-dimensional, unsteady, normed-normally distributed MARKOV-process of higher order (Schramm, 1974/75, Krippendorf, Rüdiger, Schramm, 1976). The basic simulation provides time series of monthly flows over periods of 1000 or 2000 years (or 50 runs over 50 years, for example) in order to cover the large variety of possible extreme conditions (successive years of critical low flow, series of extreme floods, etc.).

For flood periods, however, the simulation of the monthly means of flow is not sufficient because:

- the dangerous periods of floods are generally of shorter duration, particularly in mountainous river basins
- the flood control problem (see Section 1) can be solved adequately only on a daily basis.

Therefore, additional flood flow simulation techniques have been developed generating daily flows during flood periods by means of special stochastic simulation techniques. The monthly flows generated by the basic model are left unchanged in the flood simulation, i.e., the simulated long-term regime is fully preserved (see Figure 4). A more detailed description of this technique is given in other papers (Grünewald, 1977, Becker, Glos, Grünewald, 1979).

7. DERIVATION OF AN OPTIMUM LONG-TERM CONTROL STRATEGY BY INDIRECT OPTIMIZATION

One technique has been found effective for the treatment of complex large-scale WRS with reservoirs: the simulation technique allowing the evaluation of the effects of different planning and control alternatives and the selection of an optimum long-term alternative by comparing these effects (indirect optimization) (Schramm, 1976). Regarding Figure 1, the basic steps of this technique can be characterized as follows:

Taking the time series of generated flows (monthly flows normally, daily flows during floods periods) as the

- available water resources, and
- a definite initial control strategy for the reservoirs and water uses in the river basin,

an initial calculation of the resulting state conditions is

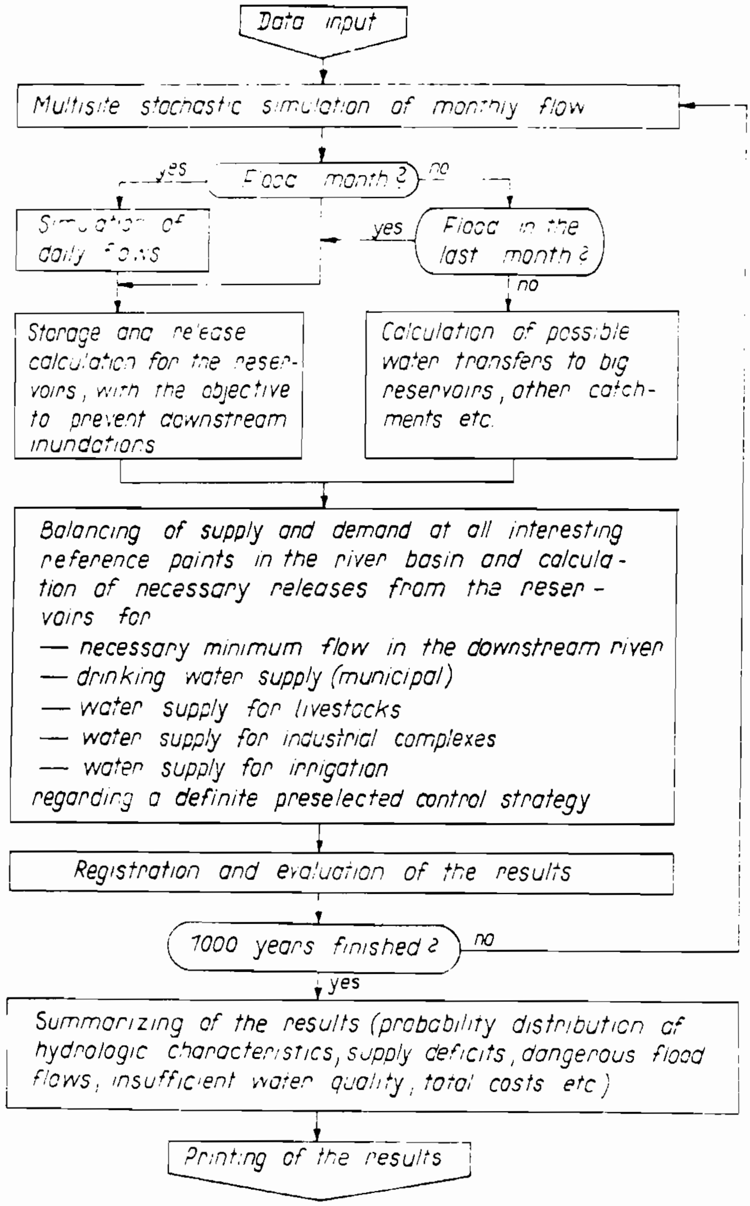

Figure 4. Flow chart for the long-term simulation of flow and reservoir control in a river basin.

carried out from month to month and

- a balance (comparison) is made with water demand (including all requirements) for interesting reference points.

If the demand, at least in some reference points, is not satisfied sufficiently, then the following alternatives are considered, or decision variables varied (see Figure 1):

- different control strategies for the controllable elements of the WRS, especially for the reservoirs;
- different water distribution principles;
- different technologies of water use and accordingly different water demand alternatives;
- different configurations of possible new elements of the WRS (additional water uses, water treatment plants, reservoirs, water transfers, levees, etc.) including the consideration of different dimensioning (maximum water stages, discharges, storage volumes, degree of water treatment, etc.) and timing of the constructions (as required).

For each alternative considered, the balancing is repeated, the water deficits, degrees of excess of the given limit values of water quantity and quality, the resulting costs, etc. are determined and registered (see Figure 4.).

In addition, frequency distributions of the interesting variables are established. The main steps of this simulation process with special regard to reservoir control are represented in Figure 4. These operations organized by a scenario (a skilled operator or a computarized strategy) are continued until the objective function is fulfilled adequately. The objective function is defined in its general form as follows (Becker, 1977, 1978):

- Derivation of a control strategy and/or of an investment alternative (i.e., for reservoirs) which ensure
  - o minimum (acceptable) material and financial efforts (minimizing the total material and financial efforts and losses in the WRS considered)
  - o a sufficiently stable water supply for all water uses to be considered
  - o a sufficient protection against floods and insufficient water quality.

The terms "sufficiently stable water supply" and "sufficient protection" are related to the "matrix of requirements" of the river basin containing all upper or lower limit values of important hydrological and water quality parameters (related to definite reference points such as river reaches, reservoirs etc.). Additionally, for the solution of optimization tasks in planning and long-term management, the precautions desired for not exceeding these limit values are given.

That control alternative (and/or investment alternative) which requires minimum costs and efforts and which best approximates the matrix of requirements is finally taken as the interesting optimum.

8. APPROXIMATIVE REPRESENTATION OF REAL-TIME CONTROL PROCESSES WITHIN THE LONG-TERM SIMULATION MODEL

According to the variation of natural flow from day to day, the monthly values of reservoir releases for flow augmentation (UM), calculated as the difference between the monthly value of the demand flow (ZM) and the monthly mean of the natural flow

(QNM) at a reference cross section would be too small. To avoid this error, a generalized function has been established on the basis of available observations (see Figure 5). It enables one to determine UM as a function of QNM (Krippendorf, Schramm, 1970).

An analogous relation is to be applied if the flow augmentation is required in order to prevent short-term excesses of given limit values of water quality parameters, e.g., of salt concentration, a certain distance downstream of the reservoir. Then the ablative effect of the river for short-term flow rises and has an additional influence.

A similar problem is given if discharge or storage dependent water transfers QT through channels or pipes of capacity QMAX from or to a reservoir must be considered. Then a relation as represented in Figure 5 must be used for the calculation of the monthly value of the transfer flow QTM.

9. STORAGE DEPENDENT CONTROL OF RESERVOIR RELEASES

In reservoir control it has been found reasonable to apply a zoning of the reservoir storage volume (see Section 5). The most important zones are (as a definite percentage of the total usable storage volume):

- the flood control volume VF serving for the retention of dangerous flood flow (with the required security). If VF is filled, at least partly, then maximum possible discharges are to be released from the reservoir (on the basis of the results of the control calculations mentioned in Section 5, point a.),
- a critical minimum storage volume VCR below which the reservoir releases U, necessary for a full supply of

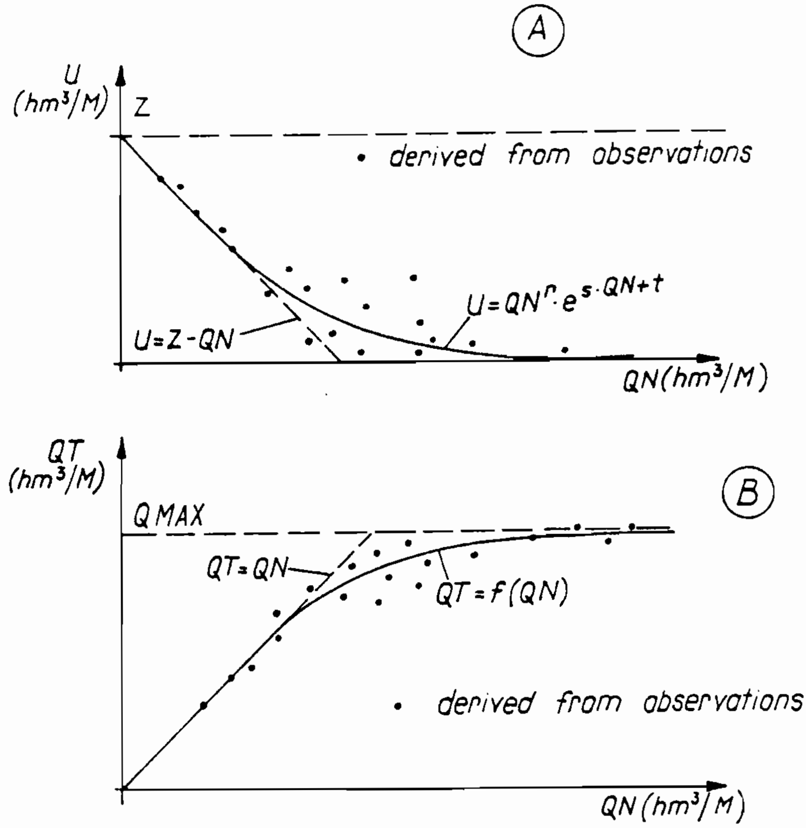

Figure 5. Monthly amount U of necessary reservoir releases for flaw augmentation (A) and possible monthly water transfers QT from or to a reservoir (B) in regard of the innermonthly variation of the natural flow QN.

all users, should be reduced by a certain definite percentage PR below the demand level (e.g., PR = 10, 20, 30%). This is useful for saving water for supply during long low flow periods.

Both storage zones may have a definite seasonal variation according to the frequency of floods and low flow periods (VF(t), VCR(t)). Within the long-term simulation runs, these parameters can be varied systematically until:

- the smallest acceptable flood control volume VF(t) is determined, guaranteeing that inundations might occur only in the case of extraordinary high floods, i.e., with an acceptable low frequency
- a critical minimum storage volume VCR(t) and a supply reduction percentage PR have beeń derived which guarantee minimum long-term losses (with special regard to long low flow periods).

Two typical results of the long-term simulation reservoir control are represented in Figure 6 and 7. Figure 6 (curve B) represents the remaining flood risk, when the flood control strategy described above (possible maximum release) and the flood control volumes VF(t) according to curve A in Figure 6 are applied for the two main reservoirs of the Saale river. Additionally, the negative influence of the reduced releases before and after the flood (0.7 of the possible maximum) is demonstrated in curve C (Becker, Kozerski, 1976).

In Figure 7, the possible flood damages and the subsequent reliability of drinking water supply from the Bode reservoir system are represented in dependence of the flood control volume

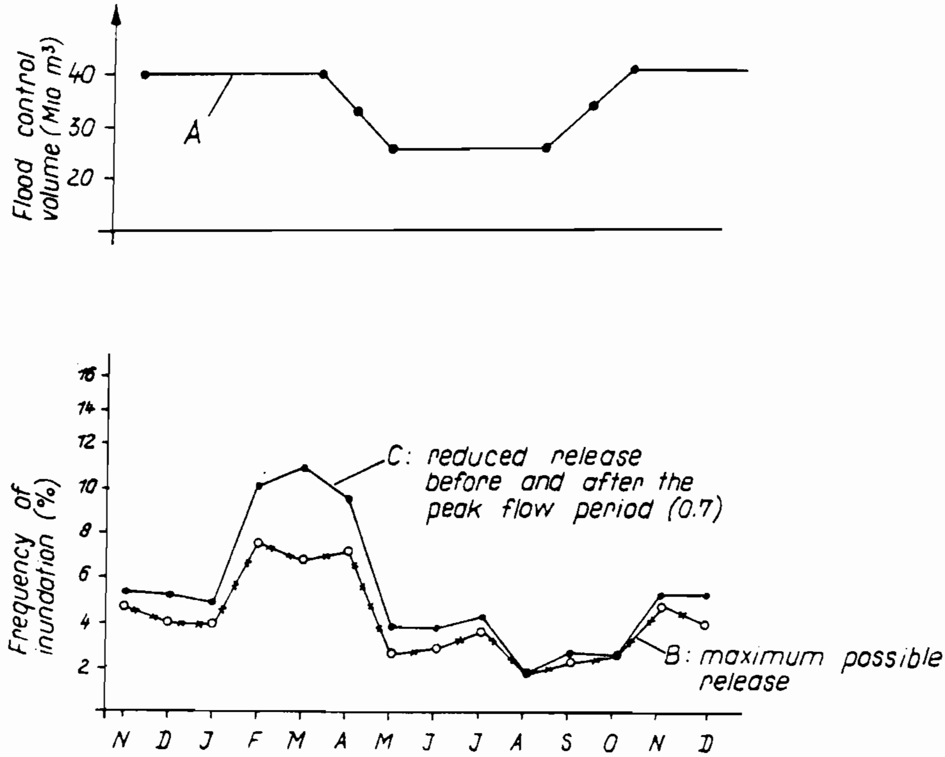

Figure 6. Longterm flood risk below the Saale reservoir system for a definite flood control volume (curve A).

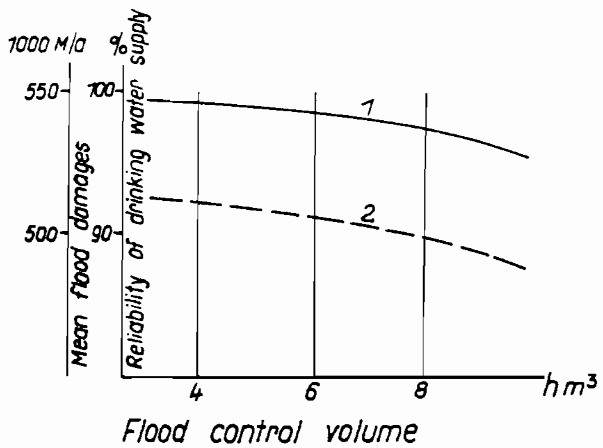

Figure 7. Reliability of drinking water supply (1) and possible flood damages (2) in dependence of the flood control volume.

of the main reservoir of the system (Becker, Krippendorf, Thiele, 1978). This figure could be taken directly for decision-making.

10. CONCLUSIONS

For an effective control of reservoir systems, it is considered necessary to apply advanced systems analysis techniques. In the GDR two types of models are applied:

- the real-time control model for short-term forecasting and control of river flow and selected water quality parameters;
- the long-term simulation model for decision-making in the planning and optimum long-term management of water resources systems.

The basic structure and some modelling principles are analogous for both model systems. The real-time control conditions of the system must be represented adequately in the long-term simulation model. The optimum control strategy derived by means of the long-term model must be taken as a basis in real-time control. The effective application of both model systems has been demonstrated in the GDR in important river basins with multipurpose reservoirs.

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OVERVIEW REPORT (FRG)

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FRG

1. Operation of Reservoir Systems in the Federal Republic of Germany

In the Federal Republic of Germany (FRG) the main water ways (navigable rivers) are operated by the Federal Government while all other river systems are operated under the guidance of Provincial Governments, which in turn delegate many of these duties to regional offices. Furthermore several river systems are operated by river authorities responsible for all activities concerning water in a particular catchment. When hydropower plays an important part reservoir operation is often in the hands of power companies.

The government agencies frequently ask private consulting firms to plan the design and operation of water resources systems (WRS). The hydraulic structures, too, are usually designed and built by private construction companies. This overview shows that in the FRG seven types of agencies are concernd with design and operation of reservoir systems:

1. Federal Ministries
2. Provincial Ministries
3. Regional Government Authorities
4. River Authorities
5. Power Companies
6. Consulting Firms
7. Construction Firms

This heterogeneous setup has advantages and disadvantages. Disadvantages are non-uniformity in design and operation of WRS^{s}, inertia in certain authorities, competition between objectives. Advantages are that new ideas are introduced from various sources, competition between system operating agencies, possibilities for competent engineers to find adequate jobs.

Due to the fact that many German firms are involved in development aid projects in various continents they have to face the competition of other internationally operating firms from many countries of the world. Therfore they have to be familiar with modern techniques, also in the field of design and operation of reservoir systems.

This way sometimes modern techniques are first applied in other countries and then only find their way into German water projects at home.

The fact that 7 types of agencies are involved in water resources is the reason why design and operation of WRS[s] in the FRG in performed on very different levels as far as the application of modern design and operation techniques is concerned. Therefore it is impossible to give an overview of the techniques applied in the FRG but rather to give some examples.

2. Some Selected Water Resources Systems in the F.R. Germany

Since there are hundreds of reservoirs in the FRG belonging to many river systems all of which are, of course, operated some way are other, it is impossible to give an overview of the operation of all these systems.

While many reservoir systems are still operated following some empirical or traditional operating rules there are several others which are operated according to modern reservoir operating policies applying techniques of systems analysis and operations research. Some of these systems which are operated this way or are being analysed with the aim to operate them in an adequate way are shown in Fig. 1.

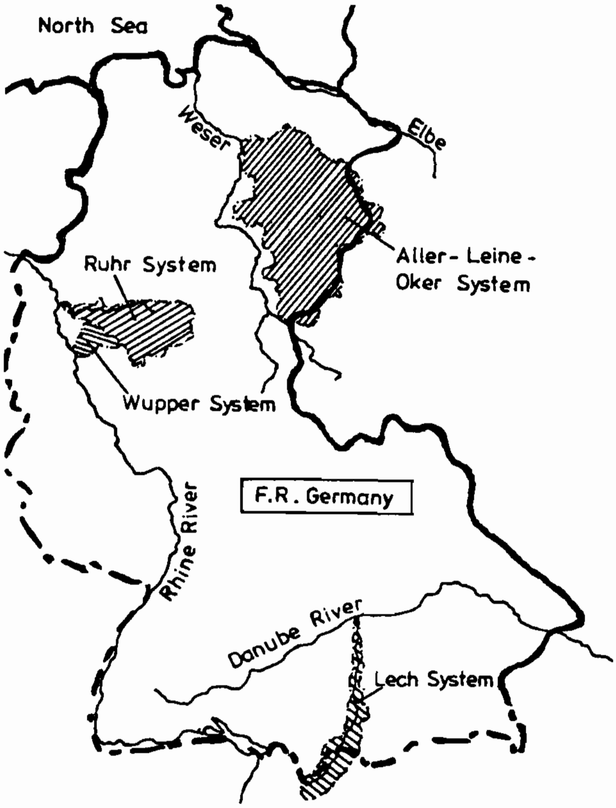

Fig. 1 Map of the F.R. Germany, Showing Some Catchments of Reservoir Systems

It may be of interest that those agencies actually administrating the reservoir systems (government agencies or river authorities) usually ask the advice of either private consulting firms or of university institutes if they decide to apply modern techniques in order to improve the operation of their reservoir systems. Some examples for cooperation between water authorities and universities are given in table 1. Chapter 4 will briefly discuss the systems and the techniques applied.

No.	System	Cooperation between
1.	Aller-Leine-Oker	Province of Niedersachsen + Techn. University Hannover
2.	Ruhr; Harz	Ruhr River Authority + Techn. University Braunschweig
3.	Wupper	Wupper River Authority + Ruhr-University Bochum
4.	Lech	Bavarian Ministry of the Interior Technical University Munich

Table 1 Examples of Cooperation

3. Operation of the Wupper Reservoir System (1,13)

As an example for the operation of a multi-unit, multi-purpose reservoir system the Wupper system was chosen (No. 3 in table 1) - not because it is better than the others, but merely because the author is actively involved in this work and therefore feels more competent here. The example is typical for a new trend in the F.R. Germany because

1. Cooperation between a river authority and a university was chosen
2. The techniques applied are presently very popular, i.e. dynamic programming and a simulation technique.

3.1 The Problem of the Wupper System

The Wupper-River-System as considered in this project comprises six reservoirs. Five of them, which are already in operation, are connected in parallel (see Fig. 2) and one, to be built, is in series with all others. Reservoir capacities range between 0,3 and 25,9 $\cdot$ $10^6 m^3$. The purposes of these reservoirs are drinking water supply, flood control, recreation and low-flow augmentation.

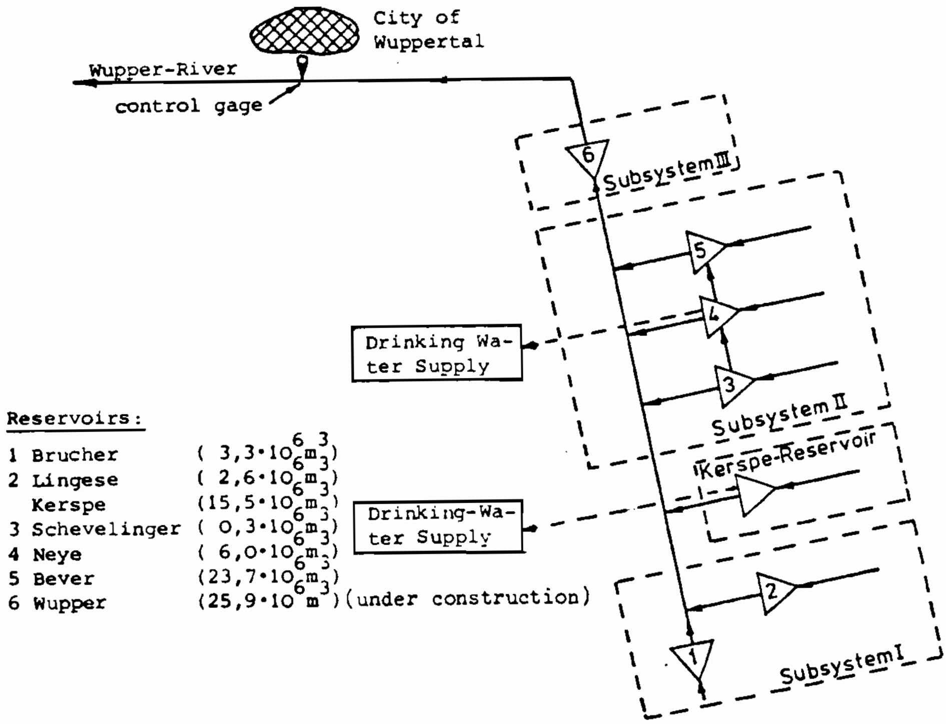

Figure 2: Wupper-River-System

The water quality of the Wupper-River is frequently very low, especially downstream the city of Wuppertal. To improve the quality many sewage treatment plants have been constructed in the watershed and the construction of the Wupper Reservoir having a capacity of $25{,}9 \cdot 10^6 m^3$, has been planned. The main purpose of this reservoir, together with the existing ones, will be to try to increase low-flow at Wuppertal as much as possible to dilute the already treated wastewaters and to improve ecological and aesthetical aspects.

Up to now the five upper reservoirs were operated trying to achieve a minimum-flow goal of 3,75 m^3/s, which was reduced to about 2 m^3/s in critical periods. This operating procedure was violated in many months during historical operation of 32 years.

3.2 The Approach Chosen for Problem Solution

The solution of an optimization model for a stochastic, multiunit, multipurpose system would be impossible given todays computer capacities and velocity, unless many simplifying assumptions on system performance are made. Therefore several authors have combined various optimization techniques or optimization and simulation. Hall (2) combines the use of dynamic and linear programming techniques whereas dynamic programming is used for single reservoirs and linear programming for the whole system. Loucks (9) uses a linear program to solve a simplified model of the system and then simulates its operation with the optimal values of the decision variables as starting parameters. Kindler (6) and Sigvaldason (12) applied a simulation model to a complex system, wherein at each step a linear program was solved to find optimal releases. The approach suggested herein has been tested by R. Harboe (3) for a simpler system and it is thought that good results can be obtained extending it to larger systems.

This approach consists of two steps, namely optimization (dynamic programming (DP)) as a screening model (first step) and then simulation (second step) which uses the optimum result of the first step as primary solutions. While the screening model works with the simplified system and process the simulation uses the real system and process. At present only the first step has been advanced far enough to be reported.

Optimization techniques are used in the first step to reduce the range of possible alternative operating rules. Optimization itself is carried out in three stages:
- obtaining single reservoir operation,
- defining subsystems, each of them consisting of several reservoirs and obtaining their operation rules
- combining the subsystems, thus representing the whole complex system and simulating their operation.

In the first stage optimum operation of reservoirs regarded as separate units is found by applying the deterministic dynamic programming technique successively to all reservoirs.

The objective function maximizes minimum flow at the control gage. Other objectives are treated as constraints or parameters that are allowed to change in different time periods.

The stochastic character of inflow to the reservoirs is implicitly taken into consideration when using both historical and synthetical samples as input to the model.

Then, in the second stage, the whole system is divided into subsystems. Several neighbouring reservoirs are regarded as subsystems (see Fig. 2). Incremental dynamic programming is used to get a higher guaranteed minimum flow than that obtained by single reservoir operation. This iterative optimization technique is applied to reduce computer time and storage as compared to standard dynamic programming algorithms (4,7). The results of the first stage are chosen as initial feasible policies which are required for the application of incremental dynamic programming. Operating rules are then derived by regression analysis and hydrologic considerations. In the Wupper -River Basin the sequence of solution of the subsystems must be taken into account, because subsystem I is solved first, and then the optimum operation of subsystem II is found considering the results of the optimum operation of subsystem I. Subsystem III is then solved considering the results of subsystems I and II.

Finally in the third stage the operating rules of the subsystems are combined and the whole complex system is analyzed at a time. At this stage of the procedure, decisions are made about priority with which the different reservoirs are emptied in order to meet the objective and the sequence in which reservoirs are called upon for this purpose. Possible criteria are state of the reservoir, inflow, level of reservoir development and evaporation losses. In this way an improvement and accomodation of the operating rules to the specific situation in the considered river-reservoir system is obtained.

The approach developed in this paper will be kept as general as possible in order to allow for its future application to any other water resources system of comparable magnitude and complexity.

3.3 First stage, Optimization

The state variable in the DP approach is the reservoir content at a time interval (here chosen to be one month).

The amount of water in the reservoir at the end of each period is chosen as decision variable. The advantage compared with the choice of release as decision variable is that there is no need for iterative procedures to compute evaporation losses, since evaporation is an explicit function of water stored at the beginning and end of each period.

The main objective is to maximize minimum flow at the control gage. The following recursive equation is used:

$$f_n(S_n) = \max_{REL_n} \left[\min \{(REL_n + Q_n),\ f_{n-1}(S_{n-1})\}\right] \qquad (1)$$

S_n : amount of water at the beginning of each period n

REL_n : release from reservoir during month n

n : periods, numbered backwards n = 1,2,....N

Q_n : flow to be augmented at the control gage

$f_n(S_n)$: optimum return (maximum of minimum flow at control gage) from periods n through 1

The state transformation equation can be written as:

$$S_{n-1} = S_n + I_n - REL_n - E_n\ (S_n, S_{n-1}) \qquad (2)$$

with I_n : inflow to the reservoir during period n

E_n : net evaporation losses from water surface of the reservoir during period n

The following constraints are satisfied during the optimization:

- mandatory release: $REL_n \geq MANREL_n$
- flood control : $S_n \leq S_{max\ n}$
- spilling : $S_{n-1} \leq CAPACITY$

As result of the optimization, a discharge level $f_N(S_N)$ at the control gage is obtained. This level can be reached with 100% probability using historical streamflow record. Synthetic inflow sequences may be applied, each of them leading to a different value of the objective function. When using synthetic streamflow sequences it must be taken into account that several streamflow sequences must be generated at a time paying attention to spatial crosscorrelation. These sequences are:

- inflow to each reservoir
- natural flow from the catchment area between the reservoirs and the control gage.

With the results of the optimization, a forward simulation is carried out, using the following operation rule: Release is maximum of:

a) mandatory releases (fish, aesthetical reasons, rights of water users)
b) release necessary for low flow augmentation up to the optimum discharge level found in backward optimization ($f_N(S_N)$)
c) release necessary to satisfy the monthly flood control reservation ($S_n \leq S_{max\ n}$)
d) release necessary to avoid spilling.

When applying this release rule there is no need for inflow forecasting because it will be possible to meet the low flow augmentation target with 100% probability when using the same inflow record (historic or synthetic) as used in the optimization. In a later stage of this research, the probabilities of reaching the target (optimization with historical record) when simulating with synthetic series will be calculated.

In the Wupper River System (see Fig. 2) first of all the Brucher reservoir is operated optimaly to increase minimum flow at the control gage in the city of Wuppertal, resulting from the catchment area between the reservoirs 1 to 5 and the gaging station. Then the program is run with Lingese reservoir considering the flow already augmented by the Brucher reservoir (Neye reservoir is not considered at this stage because of its intensive usage as drinking water reservoir). In a third run the Schevelinger reservoir increases minimum flow, already improved by the upper two reservoirs. Finally the Bever reservoir increases minimum flow at Wuppertal which already contains the releases of all upper reservoirs. This order in which the reservoir operations are optimized can certainly be reversed or any other sequence can be chosen.

Now the Wupper reservoir, which is in series with all other reservoirs, gives a chance for regulating releases from the upper reservoirs for a second time. For the optimal operation in this first step only the inflow to Wupper reservoir is the sum of optimal releases obtained from all other reservoirs plus flow from intermediate catchment. By the application of the computer program to the Wupper reservoir definitive augmented flow at the control gage is obtained.

These optimal operation policies obtained in this first stage will be used later as a starting solution (initial feasible policy) for the more complex models which will include two or three reservoirs each.

3.4 Results and Conclusions, Wupper System

From the preliminary results obtained for each reservoir only the most important one of the Wupper reservoir will be given in Fig. 3. This reservoir yields 3,37 m^3/s as optimum minimum flow at the control gage. Higher levels of flow, e.g. 4 or 5 m^3/s can be achieved with probabilites below 100% and will be simulated in the second stage. When operating the reservoirs in the suggested way, long periods with stable reservoir contents at maximum pool level are observed, which are desirable for recreational purposes.

On the basis of these results a more detailed optimization will be carried out, where groups of reservoirs will be formed and diversions between the reservoirs will be taken into account. In the second step, a simulation model of the whole complex system will be developed to improve operating rules found in the optimization part using historical as well as synthetic (multi-site) monthly and daily data as input.

4. Other Reservoir Systems in the F.R. Germany

As mentioned before, out of the many German reservoir systems only the ones given in table one will be discussed very briefly in addition to the Wupper system.

4.1 Optimization and Simulation Models of the Lech River System

The use of optimization and simulation techniques on high speed computers are very valuable tools for finding adequate operating rules. There is, however, also another empirival technique yielding good operating rules, i.e. long term experience with the prototype of a reservoir system. The following investigation showed that this technique was only about 5 % inferior to a modern optimal DP solution.

This research effort carried out at the Technical University of Munich, was applied to the Lech-River System in southern Germany near the Alps. Two parallel projects have been reported:

- In the first projekt, a stochastic dynamic programming model of the Forggensee Reservoir which maximizes the return from hydroelectric energy production in 15 power-plants was developed (5). The stationary operation rules obtained for the reservoir were applied to actual historical operation in a simulation model and improvement in energy production as compared to historical production was shown (Figure 4).
- In the second project (8) several linear operation rules were estimated through correlation using historical time series of releases, reservoir contents and inflows to Forggensee

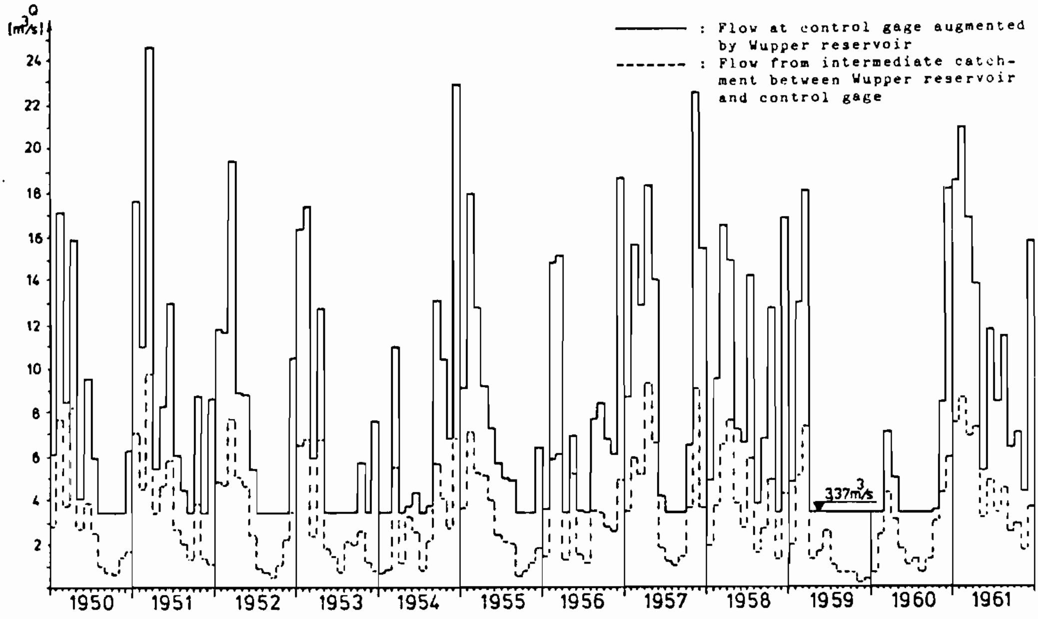

Figure 3: Flow at Control Gage Augmented by Wupper Reservoir (12 out of 32 years), Intermediate Results

reservoir. These deterministic operating policies were combined with stochastically generated synthetic inflows in order to simulate the operation of the reservoir. The best release policy, as compared to historical operation, was found to be a linear function of reservoir contens and inflow.

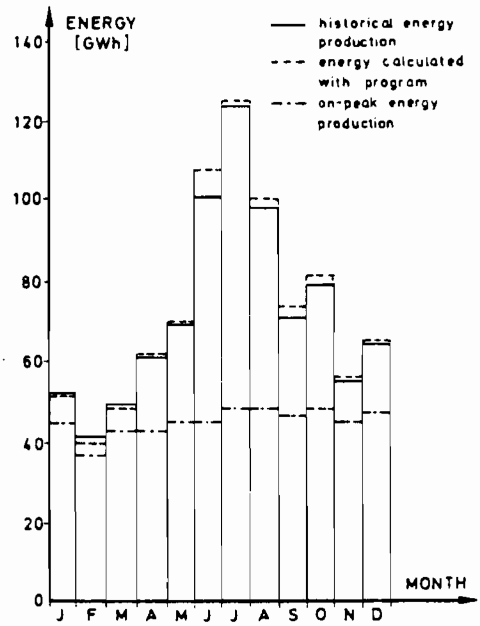

Figure 4

Comparison between Historical (actually generated) Energy and Computed Optimal Energy Production. Lech Reservoir System

4.2 The Aller-Leine-Oker Project

The main purpose of the project was flood protection in the catment of three rivers by construction of 40 flood protection reservoirs. Besides the flood protection itself the optimal sequence of construction of the reservoirs (considering budgetary constraints) was of interest. The objective function considers construction costs a well as benefits. These consist of reduction of damages to the agriculture, reduction of damages in urban areas, urbanization effects during the construction period of all 40 reservoirs, benefits from recreational effects of the reservoirs. The technique used was D.P. applied to a decision process of n steps. First step: state without reservoirs; each further step consists of addition of subsystems. All subsystems are in series thus allowing sequential decision processes.

4.3 The Systems of Ruhr Reservoirs and Western Harz

The Ruhr river reservoir system (10) and the reservoirs of the western Harz mountains (11) in northern Germany are multi purpose systems. Their objectives are low flow augmentation, water supply, flood protection and sometimes hydropower. The Ruhr system contains 5 reservoirs in the upper part of the catchment; the Harz system contains 5 existing and 3 planned reservoirs. The Ruhr reservoirs are parallel; those in the Harz mountains are parallel and in series (some are connected by conduits).

The reservoir operation mode is dependent on the season and reservoir content. Time increments of 10 days are used, the reservoir capacity is usually discretized in 9 layers. Each layer is associated with a certain release. The aim of the optimization of reservoir operating rules is to avoid spilling and complete emptiness of the reservoirs as well as not to violate constraints of given min. and max. releases. The reservoir states are simulated with historic and/or synthetic data time series. By introducing a geometric variation it is possible to optimize the operating rule for a single reservoir. In order to check for a global optimum the single reservoirs are lumped into subsystems. The results of the optimization computations were tested in the operation of the real reservoir systems and yielded good results.

5. Conclusions

In the Federal Republic of Germany reservoir systems are operated by seven different types of agencies. Thus some systems are operated in a more conventional fashion while others follow more modern systems analysis approaches. The latter is usually achieved by cooperation between a government agency or river authority on the one hand and a consulting firm or an university institution on the other.

6. Acknowledegements

The author is grateful to Dr. R. Harboe who contributed the comprehensive material for the Wupper system. Some of the other material was proveded by Messrs. Kleeberg and Maniak. The Wupper research project is sponsored by the NRW provincial government.

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THE OPERATION OF RESERVOIR SYSTEMS IN GREAT BRITAIN

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Introduction

This paper describes in brief and general terms the present usage of reservoir operating rules in Great Britain. The paper starts with a simple description of the numbers of reservoirs, their type and function, together with an indication of the organisation of water services and the ownership of reservoirs. This has been included so that the reader can relate the situation in Great Britain to that in his own country. In recent years there has been an increased interest in reservoir operating rules in Great Britain and the reasons for this are given. There are now reservoir rules in use for several purposes but at the moment most interest is focusing on rules to safeguard public water supplies; typical rules are described in some detail. The methods used to calculate operating rules are discussed and the paper ends with some brief comments on the use of short-term forecasting models and some needs for future investigation.

General Statistics

There are approximately 450 dams of 15 metres height or greater in Great Britain. The majority of the reservoirs formed are relatively small impoundments situated in Scotland, Wales and the north of England. Predominantly, the reservoirs serve a water supply function only. Approximately 20% of the 450 reservoirs are used for hydro-power generation and there are a few reservoirs which supply water to the national canal system. There are no reservoirs of any significant size used exclusively foc flood protection or for irrigation.

Although public water supply impounding reservoirs in the west or north of Great Britain form the large majority, the picture changes significantly if only the larger reservoirs are considered. There are approximately 55 reservoirs in operation or under construction with a volume of at least 20,000 megalitres. Nearly half of these are used for hydro-power generation, almost all of them in Scotland. The other reservoirs are used predominantly for public water supply and they are distributed fairly evenly throughout the country. Figure 1 shows the location of public water supply reservoirs which have, or will have a capacity greater than 20,000 Ml.

The type and role of large public supply reservoirs can be demonstrated by a classification based on the manner in which water is taken into storage and the manner in which it is discharged. Approximately 40% of the total volume stored in large water supply reservoirs is derived from gravity inflow and then discharged through aqueducts direct to demand centres. A further 30% is also derived from gravity inflow but the water stored is subsequently discharged to rivers at times of low flow to support abstraction downstream. Some 20% of the volume stored is pumped from rivers and then supplied direct to demand centres. The storage is used to provide supplies when there is insufficient flow to permit river abstraction to continue. The remaining 10% is represented by water pumped from rivers at times of medium and high flow to be discharged later back to the rivers at times of low flow to allow abstractions downstream to continue.

Most of the future growth in water consumption in Great Britain is likely to be met by abstractions from rivers, supported when necessary by releases of water from upstream surface reservoirs or from groundwater storage. The main rivers in Great Britain which receive regulation support now or are likely to in the near future are also shown on Figure 1.

Ownership of Reservoirs

a) Public Water Supply

In 1974 responsibility for water services in England and Wales and ownership of most reservoirs was given to 10 new large regional Water Authorities. These authorities are responsible for water conservation and supply, sewerage and sewage disposal, pollution control, land drainage and flood prevention, and water based recreation, amenity and fisheries.

In Scotland the reorganisation of local government in 1975 resulted in the 12 newly formed Regional Councils being given, inter alia, responsibility for water services. However, the Central Scotland Water Development Board remained as a bulk supply authority and it provides water for approximately 15% of the population of Scotland. As the years go by the Board is likely to provide all the water for growth in consumption in central Scotland. Responsibility for pollution control rests with seven River Purification Boards.

FIGURE 1
LOCATION OF LARGE PUBLIC WATER SUPPLY RESERVOIRS

In Northern Ireland responsibility for water services lies with the Department of the Enviroment.

b) **Others**

Most of the reservoirs used for hydro-electric power generation are operated by the North of Scotland Hydro-England Board who own some 80 reservoirs. Other reservoirs are operated for power generation by the South of Scotland Electricity Board and in England and Wales the Central Electricity Generating Board operate a small number of reservoirs.

Responsibility. for the canal network rests with the British Waterways Board who own some 90 reservoirs. Almost all of these are relatively small with dams of less than 15 metres height.

Reasons for Increased Interest in Reservoir Operating Rules

In the last four or five years there has been a quickening of interest in reservoir operating rules in Great Britain. Four of the main reasons are given in this section.

a) **Reorganisation of Water Services**

Prior to the reorganisations of 1974 and 1975 most reservoirs were operated by small undertakings exclusively for a single purpose, usually water supply. The amalgamation of responsibility for several water services under the control of large regional authorities has begun to result in a more flexible approach to reservoir management. Whilst the water supply function still dominates the operating policy, there is evidence that flood prevention, river water quality objectives and recreation and amenity interests are receiving more consideration. Managers are now more aware of wider responsibilities and are more prepared to be convinced that operating rules can make water available for several purposes which are not necessarily detrimental to the reservoir's main function.

b) **Joint Use**

Historically, water supplies in the west and north of Great Britain were derived from upland impoundments and supplies in the Midlands, south and east were derived from sandstone or chalk aquifers. As water consumption grew the major industrial areas had to turn to larger reservoir impoundments, often involving aqueduct transfers of water over long distances. More recently as economic dam sites have been harder to find and as aquifers have become fully committed water consumption has been met from river based supplies. It is now accepted that the yield of a region and the reliability of supplies is improved if water from one type of source is supplied at certain times and from another type at other times. Although there may be problems with blending water of different characteristics and problems caused in distribution systems, the benefits are such that most authorities now have some form of joint use scheme. The variable quantities in surface reservoirs determine when changes in supply source should be made at particular times of year.

c) **New Definitions of Water Resource Reliability**

Traditionally, the "reliable yield" of a source of water supply has been taken to be the average quantity that can be supplied continuously through a period of low runoff, such as might occur once or twice every 100 years on average. This concept is now regarded by many as being of limited value and potentially misleading. There are statistical problems associated with defining such rare events from short records of runoff. Also, a "reliable yield" cannot be taken as meaning that this level of output can be maintained in all conditions other than droughts of 1 and 2% probability. During a dry period there is much uncertainty as to its severity and no-one can predict its duration. Consequently, the demand for water has to be reduced and/or alternative supplies have to be found.

The realisation of this situation was brought sharply into focus during the severe drought of 1975/1976. This has led to the concept of expressing reliability in terms of the frequency, duration and intensity with which restrictions have to be placed on water consumption. Reliability is thus stated in terms of realistic, practical effects on the consumer. Several water authorities are now defining target levels of service of water resource reliability in terms of the expected frequency of banning the use of domestic hose-pipes, publicity campaigns to reduce consumption, water pressure reductions and the supply of water through stand pipes.

Most water authorities have recently produced reservoir operating rules to give guidance on when measures should be taken to reduce water consumption. The same rules used for the day to day guidance of the reservoir manager are also used in planning studies to provide information on the need for new resources. This need will occur when forecast increases in demands cause the frequency of restrictions in supply to reach unacceptable levels.

d) **Power Costs**

Until recently many managers expressed the view that reservoirs should be kept as full as possible for as long as possible. In the case of those sources which had to be supplied by pumping, this maxim meant that water was pumped whenever it was available for abstraction, regardless of cost and regardless of the possibility of subsequent reservoir overflow. The rapidly escalating cost of electricity supplies in recent years has caused several authorities to define rules which give guidance on when pumping should not take place, even though water is available and the reservoir is drawn down. Such rules combine hydrological probabilities and the complexities of electricity board power tariffs.

Types of Operating Rule

Although operating rules are sometimes combined to serve more than one purpose, it is possible to classify the rules now in use in Great Britain according to the function they serve. The techniques used during calculation are discussed in the next section.

a) Joint Operation - rules which define when it is advisable to take water from source A in preference to source B. The reason may be to improve the reliability of supplies or to reduce operating costs.

b) Output Reductions - rules which define when supplies to the consumer should be reduced, or when reductions in compensation water releases or prescribed flow conditions are necessary.

c) Pump Refill - rules which define when pump inflows to reservoirs can be reduced or should cease.

d) Flood Prevention - rules defining when releases from reservoirs should be made to create storage and thereby retain subsequent runoff either from rainfall or from snowmelt.

e) Power Generation - rules defining when water can be released to provide water for power generation.

f) Amenity - rules designed to hold storage at particular levels to protect amenity interests.

Figure 2 shows in diagrammatic form a set of rules which are now typical of those used for several water supply reservoirs in Great Britain. The precise form of the lines must obviously depend on the nature of the reservoir, i.e. whether it is gravity or pump filled, whether it is a direct supply or a regulating reservoir, the hydrological pattern of probable inflows, the role of other reservoirs in the system and so on. However, the zoning adopted is now quite common and merits some additional comment.

The uppermost zone A represents a normal retention level. In a gravity fed reservoir releases are likely to be made to hold storage down at this level to provide some flood retention storage. In a pump refill reservoir pumping would not take place whilst storage was in this zone and similar shaped lines below the one shown here would be used to indicate when pumping could be at reduced rates, or should occur during night time tariff periods only.

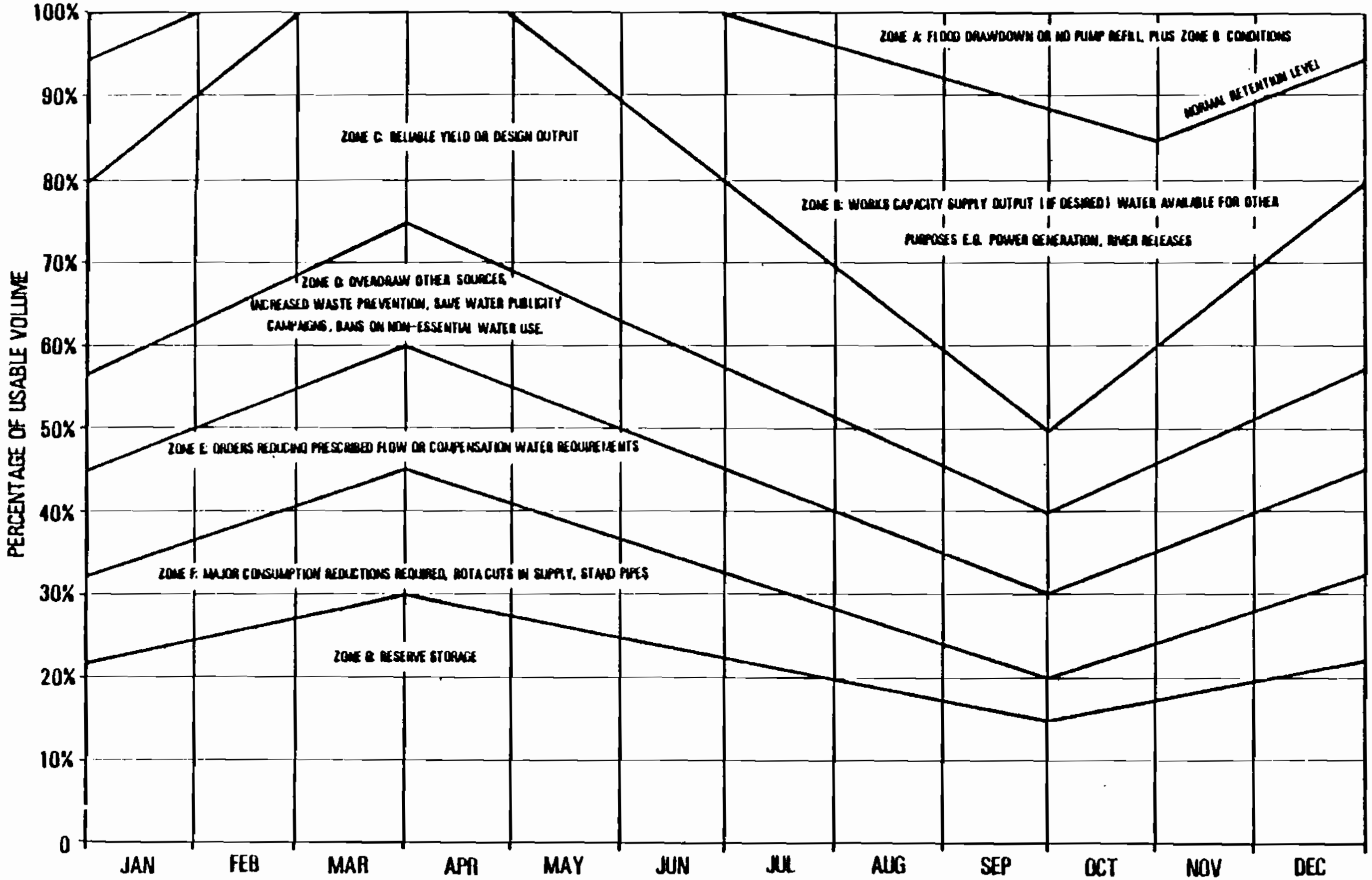
Zone B represents an area where water could be taken up to the limit of treatment or distribution capacity if this was an economic thing to do. Also, water could be released for auxilliary purposes, for example to improve river water quality or fisheries management.

Zone C might previously have been regarded as the supply available under all circumstances; a supply sometimes referred to as the design or reliable yield.

The zones at lower storage levels all represent some contraction of the normal situation. Zone D is in effect an early warning zone indicating that a particular reservoir is becoming vulnerable. Steps to be taken would include overdrawing other sources if their curves indicated that this were possible, even though such an action might be uneconomic; diverting additional manpower resources into water saving measures such as waste detection and reduction; starting publicity campaigns to ask consumers to use less water and if necessary using powers available under the Drought Act 1976 to ban water for uses which were regarded as non essential.

Zone E would indicate the need for a departure from river flow requirements. It might take the form of an Order permitting a reduction in the water normally required to be released to the river; or in the case of a pump refill reservoir, relaxation of the licence or statute controlling the amount of abstraction or time at which water could be transferred. Some authorities are attempting to make such changes an automatic consequence of a certain condition of storage, rather than using control curves merely as a guide to the need to secure the necessary Orders required to authorise such a change.

FIGURE 2: DIAGRAMMATIC REPRESENTATION OF TYPICAL OPERATING GUIDELINES FOR A WATER SUPPLY RESERVOIR



Zone F represents a very serious situation calling for reductions of 25 to 50% in water consumption. This amount of saving is only likely to be achieved by reducing the number of hours during which water is supplied through the mains network, or by installing stand pipes.

The bottom zone is sometimes referred to as reserve storage, sometimes marginal storage, and has resulted from a recognition of the uncertainty that exists during a drought as to its continued intensity and future duration. This uncertainty calls for a further element of safety over and above that incorporated into the control rules. Typically, this zone would be equivalent to 20 or 30 days output at zone F levels of supply and with an assumption of no inflow. When simulations are done using the historical flow record the rules adopted for the earlier zones will be expected to prevent storage falling to this reserve zone.

Methods used to derive operating rules

The techniques used to derive operating rules can, for convenience, be described under three headings, although it is not uncommon for a particular study to combine their use. The headings are

a) Analytical Optimisation Techniques

b) Probability Models

c) Simulation and Search Methods

a) Analytical Optimisation Techniques

These techniques use classical calculus and Lagrangian multipliers in mathematical programming to make explicit ranking of objectives. Both linear and dynamic programming have been used often in conjunction with simulation. These techniques became popular during the late 1960's and early 1970's but they are now little used. Indeed although they were used to calculate rules for several reservoirs there is now scarcely a source in Great Britain for which rules calculated in this way are in day to day use.

It seems that the dichotomy between those responsible for day to day operation of water resources on the one hand and the systems analyst and operations research professional on the other is as strong as ever. There appear to be several reasons, the main one being the difficulty of defining clear objectives which can be minimised or maximised. Reservoir operation in Great Britain is dominated by the water supply function. The risk of failure or reduction in supply has strong social and political overtones and a concensus view on value judgements on these features has not yet emerged.

Another reason is that most of the larger reservoirs and water resource systems are in an evolving state with increasing demands, additional functions and rapidly changing costs. There is therefore a need for frequent reappraisal of operating policy, objectives and control rules. Most investigations using analytical optimisation techniques have been done on a contract basis with research organisations and universities. Contract work does not lend itself to frequent reappraisal and the use of mathematical techniques has been hindered by a lack of "in-house" ability and experience. This situation is changing slowly and a few water authorities are now beginning to pick up the threads of this type of approach.

A third reason for the relative absence of analytical techniques has been the comparative simplicity of the problems. Whilst most resource systems were served by only a few reservoirs with only a single function, managers were able to evolve near optimal rules from years of experience. This situation has now altered to a large degree, partly due to recent re-organisations which resulted in significant changes in management structures, functions and areas of responsibility, partly because of changes in the resource systems and their manner of operation.

b) <u>Probability Models</u>

A number of techniques have been used to describe the stochastic nature of reservoir inflow. The transition matrix approach has been applied in several instances but the most common method under this heading is the production of inflow volumes of specified duration and probabilities. The use of one or two percent probability runoff values derived from fitting log normal or Gumbel distributions to inflow data is quite common.

A control curve derived from probability data will be used to indicate that, given a particular volume of storage at a particular time of year and the demand pattern used in the calculation (usually a constant rate of supply), there is a specified probability of storage just reaching a subsequent volume some months later. Curves are commonly calculated for a drawdown phase when the subsequent volume being guarded against is zero storage, or more recently reserve storage, and for a refill stage when the subsequent volume is the overflow level. Difficulties sometimes arise particularly during autumn in defining which phase is of greater concern.

Over the last four or five years there has been a move away from the calculation of inflow sequences of particular probabilities. Doubts have been voiced about the assumptions regarding the distribution of the data, the confidence limits that can be attached to calculations based on short lengths of data, the problems of regional probabilities rather than site probabilities and the feeling that persistence in hydrological data could be significant. Some workers believe that rules based on the worst flow sequence in the historical record, or the second worst is at least as good as a probability sequence. It is argued that probability labels give an air of precision or accuracy which can be misleading. Water authorities now seem to be evenly divided amongst those using control curves with probability values attached, those attaching a historic year label and those using no label at all but stating that a simulation of a curve of a particular shape produces a frequency of storage variation and output consequences which they regard as acceptable to their authority.

c) <u>Simulation and Search</u>

Simulation techniques are now used in almost all reservoir operation studies. Historical data sequences of typically 20 to 30 years are used to assess the response to a particular operating policy, a given level of demand, licence conditions and so on. Adjustments are made to any of these parameters and through repeated simulation this searching procedure defines a "response surface" from which a solution can be selected that is near optimal. The yardstick of what is optimal or acceptable usually has to be a manager's experience or intuition, involving as it does a subjective balance of frequencies of occurrence over a historic period, the costs of operation and the patterns of output from different sources and so on.

The frequency with which certain actions might have been necessary over the historic flow record is taken as being reasonably representative of the the frequency which might result in the future. Using the last 20 to 30 years of flow record as a guide to what will happen over the next 20 to 30 years has a high degree of sampling instability and for this reason some authorities have attempted to extend their length of flow record by using catchment modelling techniques to produce flow data from rainfall and climate data. The use of synthetic data generated statistically has been the subject of several research studies but the level of confidence in its validity has not yet been sufficient to warrant its use in reservoir operation studies.

Short Term Forecasting

Because most reservoired catchments in Great Britain are small in size and because water supply is a dominant function there has only rarely been a requirement for short term forecasting of reservoir inflows and storage fluctuations. The most notable exception applies in the River Dee catchment in North Wales where real time control of storage is a feature of reservoir operation.

There are several other areas however where real time forecasting models are being developed for catchments downstream of river regulating storage. The purpose in these situations is to improve the accuracy of river flow forecasts at downstream control stations several hours or even days ahead so that the amount of water released from storage can be made to match more closely the precise requirements as they subsequently materialise. Whilst there is currently considerable interest in these models there are as yet no real time computer based flow forecasting systems in operation in Great Britain.

Future Needs

In Great Britain there is no intrinsic shortage of water and there is only limited competition for the use of storage. Also, reservoir storage problems are concerned with smoothing out relatively brief shortages in the availability of runoff (most reservoirs are critical over a single season only). Consequently there has been little need for advanced hydrological, mathematical or programming techniques. Indeed over the last few years a number of advanced techniques have been abandoned in favour of a more simple approach. Future needs centre, therefore, not so much on techniques as on a clearer definition of objectives.

The main area of activity in operating rule development currently relates to the need for and timing of reductions in the amount of water supply normally made available to the public. In the past there has been reluctance to acknowledge that the frequency and intensity of shortages at water resource works is, to some extent, under the control of water authorities. There has been reluctance to state, in realistic terms, the reliability of a particular water resource system. This attitude is changing but it is hard to see how standards can be proposed and defended until more analysis has been done on the benefits of an uninterupted water supply, the benefits of having water available to meet new developments, and the costs of interruptions in supply of varying frequencies, durations and intensities.

There is also a need for more analysis of the benefit or cost of 'in situ' river requirements for activities such as fisheries management, navigation, amenity and flood reduction. Without some move towards a cost benefit type of study it will be difficult to make optimum allocations of storage.

In the short time which has elapsed since the re-organisation of water services in Great Britain many water resource systems have been made more reliable and enhancements are being planned on a more rational footing. However, apart from a general levelling up and standardisation of approach it seems likely that the calculation and day to day use of reservoir operating curves will not extend beyond the methods and usage outlined in this paper until the objectives of reservoir management have been defined more clearly.

OPERATION OF MULTIPLE RESERVOIR SYSTEMS IN FINLAND

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The optimal operation of multi-reservoir systems is currently one of the basic problems in developing water resources in Finland. This is partly due to the need to revise old operation rules, either because the objectives of the systems are changing or because they have to be stated more explicitly. It has also been generally understood that modern computers and optimization methods would be able to handle even very complicated systems.

There are 55,000 lakes in Finland with a total area of 31,000 km^2. This accounts for about 9 per cent of the country's total lake area. If these lakes were evenly distributed over all the watersheds there would be excellent natural regulation. Unfortunately, this is not the case. Over 70% of the total lake area is in the Lake District, which covers only about one third of the country. On coastal areas and in northern Finland, natural regulation is insignificant, and the need for reservoirs and efficient operation schemes is obvious.

During the last twenty years, about 30 reservoirs were built in Finland. At the maximum water level, their surface area is 930 km^2 and active storage is over $2{,}900 \cdot 10^6$m^3. Several new reservoirs are in the planning stage, including two large ones (220 and 210 km^2) in northern Finland. In relation to the surface area, about 70% of the existing reservoirs have been built mainly to facilitate hydropower production. Another important purpose is flood control, which is especially important in the flat river valleys of western Finland.

In addition to the construction of reservoirs, many lakes are also regulated. The area of lakes now under regulation is over 10,000 km^2 and active storage is about $22{,}000 \cdot 10^6$km^2. Thus the active storage of regulated lakes is 7.5 times larger than that of man-made reservoirs. In many cases, the natural variability of water levels is only slightly increased by regulation. Their annual patterns have however been changed to coincide with the demands of water users.

Regulated multi-reservoir systems exist both in the Lake District and in the coastal and northern parts of the country. The typical operation schemes are obviously quite different. In the Lake District, it has been traditional to formulate a scheme individually for each lake. This is partly due to the fact that different organizations operate different power plants, and partly because the system has generally been considered to have an abundant storage volume. The effects of upstream regulation on the lower reaches of the lake systems have been considerable, but not especially harmful. However, there have been minor conflicts of interests, especially between hydropower, agriculture and fishing. Despite the significant differences between various operation policies, all have one basic feature: the reduction in the water level during late winter in order to create enough storage for snowmelt floods.

In coastal areas and in northern Finland, storage volumes are much smaller than in the Lake District. It usually corresponds to only part of the volume of spring flood, which may represent 30-50 per cent of the annual discharge. In most cases, a considerable drawdown of the water level is made during winter. During flooding, the releases are determined according to the following criteria:

-- harmful releases should be avoided
-- harmful water levels should be avoided
-- reservoirs should be as full as possible at the end of the flood.

Close cooperation between different regulating authorities is essential if these goals are to be attained. This is usually tried without sophisticated calculations, sometimes even without the use of any mathematical model, but using engineering judgement based on the experience gained during previous floods. This is often sufficient in the case of "normal" floods, but exceptional situations are difficult to handle and losses are likely to occur.

In summer, heavy rains may sometimes fill the smaller reservoirs, leading to spillage. However, this has been considered a problem of secondary importance, although the events of summer 1974 caused a reconsideration of this attitude. It is more important in summer to determine the releases, so that exceptionally low water stages can be avoided.

Sophisticated mathematical methods have usually not been applied in planning rules for operating reservoir systems. The exceptions include the dynamic programming model of the Helsinki University of Technology, which is capable of handling up to four reservoirs. An out-of-kilter algorithm has also been used to determine the operation policy of a multi-reservoir system. This has been done in conjunction with a branch-and-bound algorithm, which solves the capital budgeting problem if some alternative reservoir sites exist in a planned reservoir system.

Much more effort has been directed to inflow forecasting than to the determination of rules for multi-reservoir operation. The water equivalent of snow cover is the most essential piece of information, and for this reason an extensive network for snow observations has been established in Finland.

The most commonly applied inflow forecasting method is multiple-regression analysis. Besides the water equivalent of snow cover, several other hydrometeorological variables (precipitation, temperature and previous inflow) are used. Most of the forecasting schemes are monthly or seasonal. In a few cases, models have been calculated for each month or season, but usually these are only needed for the snowmelt period. The first estimates of snowmelt flood volume are sometimes given as early as January. The maximum water equivalent of snow cover occurs in March-May, depending on the latitude and weather conditions, and the date of the "final" forecast is usually April 1 or May 1.

Under average conditions, the regression forecasts can be considered satisfactory. When the snow cover is heavy, the melting period short or when rainy weather prevails during melting, the forecasting error may increase significantly. In such cases, pure judgement can give better results than regression models.

The main emphasis of inflow forecasting is currently on the application of hydrologic simulation models. The first attempts have been quite promising and work is continuing in this field. Different types of models should be developed for the Lake District and for the coastal areas. The development of these models and their use in connection with new operation models could obviously solve some of the problems currently limiting the efficient use of waters in Finland.

OPERATION OF WATER RESOURCE SYSTEMS IN CZECHOSLOVAKIA

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INTRODUCTION

The methods of determining an operation policy and its application are unique for each water resource system in the CSSR. Therefore, they will be discussed in the overview of the following five water resource systems that are now in operation:

1. Water resource system of the River Vltava (Vltava Cascade)
2. Water resource system of the River Váh (Váh Cascade)
3. Water resource system of the River Ohře
4. Water resource system of the River Odra
5. Water resource system of the River Dyje.

1. THE VLTAVA CASCADE

This system consists of three main reservoirs in series, i.e., reservoirs Lipno, Orlík and Slapy and four smaller reservoirs (see Table 1). The system was completed with construction of the dams Orlík and Kamýk in 1963.

The operating policy of the system was determined mainly by the primary aim, i.e., electric energy generation in water power plants. However this multipurpose system has additional aims.

Table 1. Reservoirs of the Vltava Cascade.

Reservoir	Storage mil. m^3 Dead	Active	Relative+/	Total	Drainage Area km^2	Mean Annual Flow $m^3.s^{-1}$
Lipno I.	23	252	0.59	306	950	13.6
Lipno II.	-	1.6	-	1.7	996	13.7
Orlík	280	374	0.14	716	12106	82.5
Kamýk	8.3	4.4	-	12.8	12217	82.9
Slapy	68	184	0.07	269	12957	84.9
Štěchovice	6.4	4.7	-	11.1	13298	85.5
Vrané	8.6	2.5	-	11.1	17782	111.0

+/Relative storage = Active storage/Mean annual runoff

It ensures the river flow regulation downstream of the cascade in Prague up to 40 $m^3.s^{-1}$. This value is necessary to maintain the required quality for the municipal water supply in Prague. Before regulation, the minimum flow was approximately 10 $m^3.s^{-1}$ (the catchment area of this site is approximately 27,000 km^2 and the mean annual flow is approximately 150 $m^3.s^{-1}$).

A further objective of the system is the delivery of water for irrigation downstream of Prague, in the lower part of the Vltava drainage basin. The maximum demand for irrigation is approximately 4.2 $m^3.s^{-1}$. As the required minimum flow downstream of Prague is about 23 $m^3.s^{-1}$ this requirement for irrigation is met when the required flow in Prague is achieved.

Yet another purpose of the Vltava water resource system is recreation. The Slapy reservoir was built for this purpose. It is relatively near to Prague (about 50 km). It is large enough to create the conditions for temperature stratification of water in summer (with a warm upper layer, the water is clean) and last but not least, the requirements for recreation--a steady water level in summer (within 1m tolerance limits) which does not conflict with the demands of power generation.

The method used for determining the operating policy was rather straightforward and was a modification of techniques proposed by Hufschmidt, Fiering (1966). First, the critical period 1933-1935 was analyzed and then the rule curves derived for reservoirs were tested in the period 1931-1960 by a deterministic simulation model. The rule curves were adjusted by marginal analysis and these new rule curves were tested by a stochastic simulation model on the basis of 500 years of stochastic hydrology, and again corrected. The monthly operation policy was and is determined by these rule curves.

The short-term operating policy differs according to the flow conditions. In a period without floods, the constant mean daily power output (W) is the aim of operation. The values of W are dependent on the day of the week and the season and the rule curves were calculated on the basis of the values of W. It was a difficult task, as the water capacities of turbines and the relative volumes of reservoirs in the system were different. For instance, the Lipno I reservoir has a long-term cycle with carry-over capacity, while reservoirs Orlík and Slapy have one year (or two year) filling and release cycles. Therefore, the coordination of operating policies of these reservoirs in the system requires that the Lipno reservoir be filled first. Water release from the Lipno reservoir passes through the whole system and generates (especially when the reservoirs are full) the greatest power output.

It was not possible to find an analytic, mathematical solution to this problem of cooperation between reservoirs. Therefore, the methods of engineering judgement, critical period analysis together with experiments made with simulation models were used. The problem was further complicated when the system was extended by the Váh cascade. The cooperation between two hydrologically different subsystems leads to a higher reliability of the system's function. However, the analysis was carried out with the aid of a deterministic model, with the aim of obtaining the constant reliability of the mean daily power output of the whole system, including the Vltava and the Váh cascades.

The operating policy was improved by marginal analysis. On the basis of the constraints created by the other aims of the water resources system of the Vltava and the Váh catchments, the objective function was maximized. This objective function was evaluated from the standpoint of power generation only. The method used offered a very good solution when the primary objectives were water power generation, river flow regulation and recreation, and when flood control was considered an aim of secondary importance. However, if this is not the case, the system operation policy needs further analysis.

In the Vltava water resource system, the probability of extreme floods is nearly independent of the season or month. The occurrence of extreme floods is dependent on the synoptic meteorological situation, in combination with the runoff conditions. The flood control storages in reservoirs are however constant. Therefore, they are inadequate for the control of extreme floods. For this purpose, the entire volume of the Vltava cascade would be necessary which of course is not possible in a multipurpose system. Therefore flood control of the River Vltava in Prague is for a flood of approximately 10% probability (10 year recurrence interval). Without the flood control storage of the Vltava cascade it would be 20% (5 year recurrence interval). After reconstruction of the weirs in Prague, the short-term operation policy of the Vltava cascade will enable control of floods exceeding 5%. Data on floods are shown in Table 2. The observed flood in 1880 had the largest volume--1 km^3 with a peak flow of 2500 $m^3.s^{-1}$.

The floods of higher recurrence intervals are the consequence of a combination of a certain synoptic situation (the movement of the polar front from the South to the North) and the favorable runoff conditions (e.g. after heavy rains when the soil is wet). This situation forebodes a flood but the peak flow and the volume of the flood is difficult to forecast. The decision when to release water from reservoirs is dependent on this forecast and

Table 2. Floods in Prague (without flood control)

Recurrence interval years	1	2	5	10	20	50	100
Peak discharge $m^3.s^{-1}$	730	1130	1790	2279	2840	3600	4900
Volume of the 5-day flood mil. m^3	122	167	232	281	330	406	467
Volume of the 10-day flood mil. m^3	213	288	402	486	576	713	823

is therefore a difficult task. The new meteorological and hydrologic forecast system with the Prague radar station forms the basis of the forecasts of better quality and longer forecast intervals. This system will enable a better short-term operating policy for the Vltava cascade.

The problem of flood control is investigated not only from the statistical point of view but also in relation to the long-term hydrological and meteorological cycles. During this century and during the operation of the Vltava cascade floods of middle recurrence intervals have occurred. According to the long-term forecasts of some hydrologists, greater floods can be expected at the end of this century, as some long-term cycles will coincide and the flood risk will grow. However, it is probable that by this time, the above mentioned measures will be put into operation and through the multipurpose operation of the River Vltava System, floods will be controlled.

From the methodological point of view, an interesting problem is the investigation of the future operating policy of the system in connection with the possible new aim of the system--utilization of the reservoir Slapy as the new water resource for municipal water supply. The main elements of this system are the waterworks in Prague with poor water quality and the reservoir Želivka (see Figure 1) with water of better quality. The yield

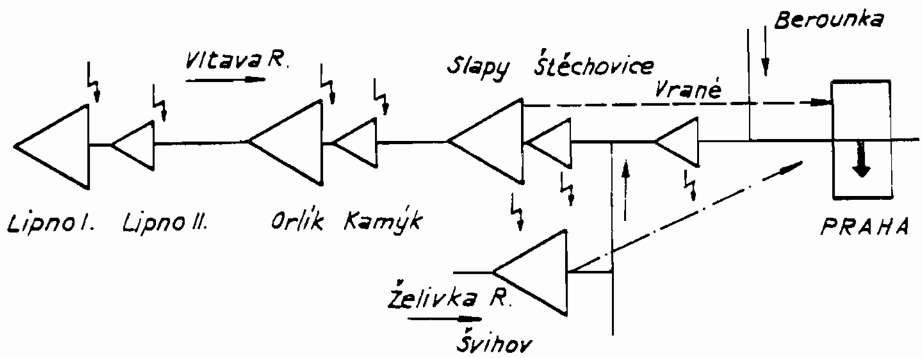

Figure 1. Schema of the Vltava water resources system.

of this reservoir can be enlarged by a transfer of water from the River Sázava to the River Želivka but it is questionable because of the variable water quality in the River Sázava. Another possibility is to withdraw water from the reservoir Slapy with lower variability in water quality (as the level of withdrawal can be controlled) but with an impact on the operating policy of the Vltava cascade. The water resources have different costs and different and variable quality and quantity of water. For determining the best alternative, the Stochastic Model of Alternative Determination--SMAD--is investigated (the formulation of this model is given in Appendix A).

The operation of the Vltava cascade is carried out by the energy and water management operating boards. The short-term operation under normal conditions is carried out by the energy operation board. It is centralized and the power plants are operated from the central body in Prague.

2. THE VÁH CASCADE

The primary target of this water resources system is also power generation, therefore the operating policy is similar to that of the Vltava cascade. It consists of three main reservoirs (see Table 3). The secondary aims are the delivery of water to irrigation systems in the lower part of the River Váh catchment and the River Nitra catchment and recreation in the main reservoirs. Flood control is a secondary aim. Unlike the Vltava cascade, many power stations are located on the diversion canals (see Figure 2). This was forced by the transportation of gravel in the river bed, however additional problems of the unsteady flow in canals were created and solved.

Table 3. Reservoirs of the Váh cascade

Reservoir	River	Storage mil. m^3 Active	Relative	Total	Mean Annual Flow m^3/s	Drainage Area km^2
Ustie	Orava	298	0.46	347	20.7	1181
Liptovská Mara	Váh	320	0.36	360	28.2	1493
Nosice	Váh	24	0.01	36	130	7890

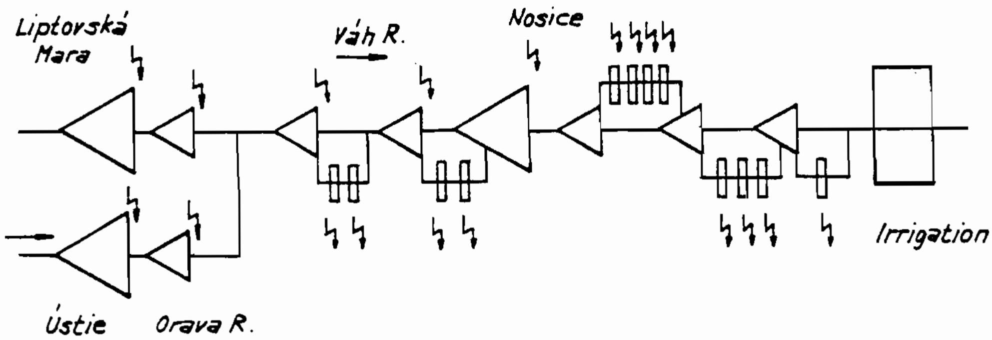

Figure 2. Schema of the Váh water resources system.

Methodologically, the operating procedure determination was similar to that of the Vltava cascade. This was an advantage in the connected system of the Vltava and the Váh cascades. The flood control objective was the main difference, as the River Váh catchment is designed for protection of the agricultural area. Therefore, the lower degree of protection is in accordance with the possible flood damages.

3. WATER RESOURCE SYSTEM OF THE RIVER OHRE

This system consists of two subsystems, i.e., the subsystem of reservoirs in the catchment of the River Ohře (see Figure 3) and the subsystem for flood control for the coal-pits and of reservoirs on small rivers that cross the boundary. These reservoirs have small drainage basin areas (see Table 4) and therefore they are long-term reservoirs. They serve predominantly for public water supply to Northern Bohemia together with the reservoirs in the River Ohře drainage basin.

Now the operations on the main watercourse of the system, the River Ohře, follow the rule curves. The operation of the

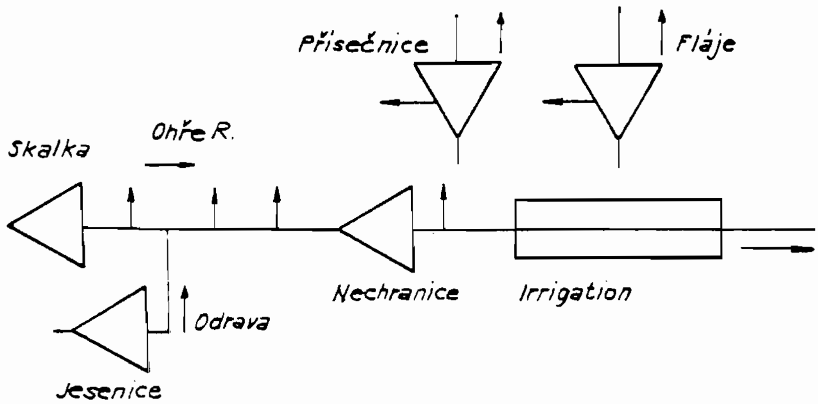

Figure 3. Schema of the Ohře water resources system.

Table 4. Reservoirs in the Ohre W.R.S.

Reservoir	River (Brook)	Storage mil. m^3 Dead	Active	Rel.	Total	Drainage Area km^2	Mean Annual Flow m^3/s
Fláje	Flájský brook	1.8	19.5	0.81	21.6	43.1	0.76
Přísečnice	Přísečnický brook	2.8	46.7	2.28	50.6	46.2	0.65
Skalka	Ohře	0.9	15.0	0.08	15.9	672	6.09
Jesenice	Odrava	0.2	50.0	0.49	52.2	406	3.25
Nechranice	Ohře	2.6	233	0.24	272	3590	30.8

Fláje and Přísečnice reservoirs is accomplished for the given draft. For these reservoirs the main problem of the draft calculation lies in the determination of the mean annual flow at the reservoir sites as there are no measured data series of adequate length and the values of the mean annual flow determined by the hydrologic analogy are in the interval (0.7-0.9) for the reservoir Fláje and similar values apply for the reservoir Přísečnice. With this example of operating policy the importance of the input data of the model can be shown. For instance, by very sophisticated methods, a gain of 10% can be achieved, as compared to the difference due to input data that can be as high as 30%.

The analysis of the operating policy of these reservoirs reveals that this problem is far from being solved. In a simulation model, the duration of the critical period can be determined, i.e., the period when the reservoir will be fully utilized even in case of variable draft caused by cooperation between the system's components. In real operation, it is difficult to define in which part of the dry period the system actually is. Therefore, the carry-over reservoirs are mostly operated on the constant draft.

In the special case of reservoir Fláje and Přísečnice co-operation on the operation up to the final stage is carried out. This operation enables releases of as much as 120% of the calculated draft from the Fláje reservoir for a period shorter than the critical period. These higher values will be changed to lower values (e.g. 80%) when the reservoir Přísečnice is put into operation. This is possible because the full draft of both reservoirs is lower than the demand for water. This operating policy is adjusted to the dynamic growth of the system demands and secures the same final state of the reservoir Fláje as if it were operated on the constant draft. This operating policy is dependent on the coincidence of this period with the flows of medium values that are necessary for the first filling of the reservoir Přísečnice. The risk connected with such an operation is analyzed.

The reservoirs Skalka and Jesenice serve as a means of river flow regulation in the middle part of the River Ohře catchment from Chomutov as far as Nechranice, especially for the demands of industry. The reservoir Nechranice serves further industrial demands and the demands of irrigation. As the development of irrigation proceeded more slowly than assumed, a part of the Nechranice active storage is not utilized for irrigation and can be therefore used for flood control and in winter for control of temperature in the river downstream to prevent ice formation.

Now the main problems of this catchment are connected with environmental management and coal mining. For the operating policy and system development protection of mines against floods is of prime importance. This is because of the abandonment of the reservoir Dřínov due to hydrologic changes brought about by deforestation, in turn caused by the emissions (namely SO_2) from thermal power stations. For this purpose, channel improvement has been designed.

Further problems may be created in the future by the temporary interruption of reservoir Jesenice's operation, if the resources of coal under this reservoir are mined. The risk determination for this type of operating policy needs special analysis and there has been no precedent.

4. WATER RESOURCE SYSTEM OF THE RIVER ODRA

The main purpose of this system is the delivery of water for municipal water supply and industry. The additional aims are flood control and recreation. The demand of irrigation and primary power generation in water power plants are of lower importance. The system of five main reservoirs is now in operation (see Table 5). The growing demands in this catchment area will require the enlargement of the system by the reservoir Slezská Harta in the near future (see Table 5 and Figure 4).

Table 5. Reservoirs of the River Odra W.R.S.

Reservoir	River	Storage mil. m^3 Dead	Active	Rel.	Total	Drainage Area km^2	Mean Annual Flow m^3/s
Kružberk	Moravice	4.0	20.0	0.11	35.6	556.7	5.95
Žermanice	Lučina	1.0	18.5	0.51	20.1	45.4	1.15
	with the transfer of water from the Morávka River			0.16		132.1	3.57
Těrlicko	Stonávka	0.6	22.0	0.61	24.3	81.6	1.14
Morávka	Morávka	0.4	4.4	0.08	10.1	63.3	1.77
Šance	Ostravice	2.5	45.8	0.47	54.2	146.3	3.11
Slezská Harta	Moravice	10.0	200.0	1.19	210.0	464.2	5.35

The determination of the operating policy of this water resources system is methodologically interesting. As the water balance in the catchment is negative in the drought periods and there is a substantial danger that the system could fail to meet the demands, many methods were investigated in order to strengthen the system till the reservoir Slezská Harta is put into operation Other investigations were made to find out how the Slezská Harta reservoir could be optimally incorporated into the system.

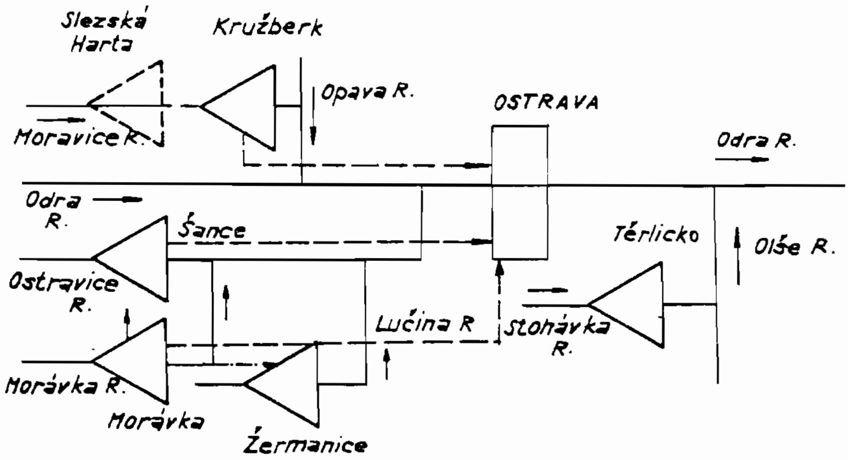

Figure 4. Schema of the Odra water resources system.

Even if the main aim of the system is the delivery of water to municipalities and industry, during the summer months the recreation levels in the reservoirs Těrlicko and Žermanice have to be maintained. Therefore the calculation of the risk that some of the demands will not be met was done by the method of "multimodelling" and included the following methods:

(a) chance-constrained model with the linear decision rule,

(b) chance-constrained model with the direct draft optimization,

(c) deterministic simulation model with monthly flows from the period 1931-1970,

(d) stochastic simulation model with synthetic hydrology using the principal component analysis combined with the central station model,

(e) "release-maximum" model,

(f) combination of simulation model with chance-constrained model,

(g) flow in networks model (under investigation).

The theoretical guidelines for the operating policy of the system were the results of these models. The application and implementation of these results were done only partly. The obstacles were formed by the water rights of the consumers and the difficulty to make the operation of individual reservoirs more flexible. Some consumers have the possibility of withdrawing water of different quality from different resources. The optimal operating policy forces them in some cases to take water from the resources of poorer but adequate quality so as to make possible the future withdrawal of water for other uses. In some cases there is no legislative measure to force these consumers to act in this (desirable) way. For an evaluation of the consequences of this consumer behavior, a simulation model with real operating policy (based on the results of nonantagonist games with cooperation) was calculated in order to find the real risk of system failure.

In the simulation model of optimal incorporation of the reservoir Slezská Harta into the system, the method of synergism (Hirsch, Cohon, and ReVelle 1977) was used. Unlike the case discussed by these authors, the three main reservoirs in the system that are determined primarily for public water supply have different relative storages and they can be operated in the system in a different way. Therefore the gains in the monthly intervals were substantial (Hirsch, Cohon, and ReVelle state the gains from the daily operation only and not from the monthly one). The details of the stated model are given in the case study by Kos and Zeman, The Odra River Water Resource System (this volume).

5. WATER RESOURCES SYSTEM OF THE RIVER DYJE

The main aims of the water resource system of the River Dyje are the delivery of water for public water supply, for irrigation, flood control, power generation, environmental management and recreation. These goals are of equal importance and therefore the multipurpose aspect is the main feature of this system. It consists of five main reservoirs. The reservoir Dalešice has the pumping water power plant but its reservoir serves not only for power generation but other purposes, too (see Figure 5). However,

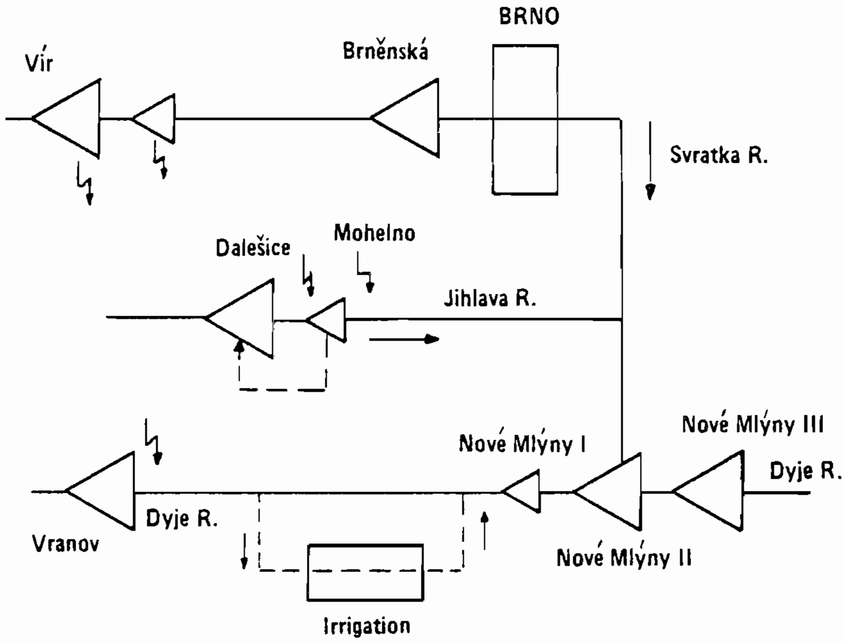

Figure 5. Schema of the Dyje water resources system.

due to the lack of energy, the tendency has been to strengthen power generation. The reservoir of Nové Mlýny consists of three reservoirs. The reservoir Nové Mlýny III serves for flood control and delivery of water for irrigation, the reservoirs Nové Mlýny I and II were built for environmental management, flood control and recreation (see Table 6). The reservoirs Nové Mlýny are relatively shallow and during operation vast areas could become moors, which would bring about further environmental problems. Therefore the two upper reservoirs have a constant water level (with the exception of floods). As the area of these reservoirs were flooded very often, the flood control storage reduces the environmental problems of this part of the catchment.

From the methodological point of view, the subsystem of the upper and middle part of the River Dyje catchment is interesting. The reservoirs Vranov ensures river flow regulation for the diversion canal Krhovice-Hevlín. A minimum flow should be maintained there, regardless of the withdrawals for irrigation. As the

Table 6. Reservoirs of the River Dyje W.R.S.

Reservoir	River	Storage mil. m^3 Dead	Active	Rel.	Total	Drainage Area km^2	Mean Annual Flow $m^3.s^{-1}$
Vír	Svratka	13.3	34.6	0.30	53.1	414	3.7
Brněnská	Svratka	7.6	10.9	0.04	18.5	1575	8.0
Dalešice	Jihlava	59.5	67.8	0.36	127.3	1136	6.0
Mohelno	Jihlava	5.6	11.4	0.09	17.1	1155	6.0
Vranov	Dyje	31.5	80.0	0.24	122.7	2221	10.6
Nové Mlýny I	Dyje	7.5	4.5	0.01	12.0	4557	14.4
Nové Mlýny II	Dyje	17.2	8.5	0.01	33.5	11720	41.7
Nové Mlýny III	Dyje	25.2	52.0	0.04	94.4	11825	41.8

demands for supplementary irrigation are very variable and depend on the meteorological and hydropedologic conditions of the irrigated lands, losses due to unused canal return flow occur; the problem of minimization of these losses is a stochastic one and it was the subject of many investigations. For instance in the catchment of the River Dyje, the combination of the simulation model and linear programming with continuous and discrete (bivalent) variables was used (see Appendix B).

This model was applied for the optimization of the operation policy of the system that consisted of two subsystems, i.e., the water resources subsystem and the agriculture-irrigation subsystem. As this latter subsystem described the agricultural production on irrigated lands in detail, it was necessary to shorten the analyzed period to a few years. Therefore a step-by-step method was used. The first part of the investigation was formed by simulation with a coarse agriculture-irrigation model. The aim of this simulation was the determination of the critical period of several years that was then used in the detailed agriculture-irrigation model. However, the inputs of the linear model differ from that of the simulation model. The outputs of the system were also different.

Therefore, the procedure of simulation modelling with operating policy determination and its application in linear programming was repeated several times to achieve an overall optimum for the system. Although all the aims of the water resources system were taken into account, the optimization was done from the standpoint of irrigated agricultural production and the other aims were taken as the costs and constraints. To take into account all the aims on an equal basis, a multiobjective evaluation procedure of decision analysis called Fuller's method was investigated.

6. SUMMARY

In Czechoslovakia, there are five water resources systems that can be characterized as multireservoir and multiobjective systems. Therefore, for this overview, the method of their description and analysis was used with the accent on their specific features, either from the standpoint of aims and configuration or methodology. From the latter point of view, only four types can be considered, as the Vltava River and the Váh River cascades have common properties and in some models they were investigated as being in one system.

The main progress in operating policy analysis lies in the method of "multimodelling," when the same problem is solved with different models with nearly the same inputs but under different hypotheses. It is common practice in modelling to change the values of the model input parameters and seek the output response (e.g., the change in the value of the risk in fulfilling the individual targets of the system), in other words, to experiment with the model.

If the fact (1) that the model is a reflection of reality is realized, (2) that it is dependent on some presumptions and hypotheses, (3) that it expresses some theories, and (4) that it has to simplify reality, then it is clear that the application and implementation of the results given by one model are not without risk. The changes of the values of the model input represent parametrization of the model. However, they are not concerned with the basic presumptions, hypotheses and premises.

The basis of the multimodelling is the series of premises which allow the models both to be represented and to perform less rigidly. In determining operating policy, this procedure leads to different premises about the function of water resources systems in the national economy.

In this process of multimodelling a decisive role was given to simulation models, as they best reflect reality. Therefore, the simulation models were used for the verification of all the alternatives of the operating policy derived in models of operations research. This combination of simulation models and models of operations research was chosen as the simulation model alone (and especially the stochastic simulation model) is a clumsy instrument for optimization. The search for the optimal alternative is attained by repetition of the calculations, which is time consuming for both computers and people.

In a combination of simulation models and models of operations research, the variety of inputs is limited by operations research models and the analysis by simulation models can be done mainly in the optimum area and the target can be arrived at in a more effective way. To use this combination to advantage, the models of operations research applied in this way should be sophisticated but simple, i.e. for the model construction a sophisticated method can be used but the way it is applied should be simple (the input values of such a model should be relatively easy to obtain).

For this reason, the practice in some water resources systems, e.g., in the River Odra and River Dyje catchments, and further theoretical research in the CSSR aims at the effective combination of the methods of design and operation of water resources systems in the system of multimodelling.

APPENDIX A: STOCHASTIC MODEL OF ALTERNATIVE DETERMINATION--SMAD

Model SMAD can be utilized under the following conditions: The i^{th} water resource can be used or constructed in J alternatives. The indices of the alternatives are $j=1,2,\ldots,J$. The variable y_{ij} attains the values 0 or 1 (bivalent variable), $y_{ij} = 1$ when the i^{th} resource is used in the j^{th} alternative, otherwise $y_{ij} = 0$; therefore

$$\sum_{j=1}^{J} y_{ij} = 1$$

The problem of water resources system utilization is stochastic and therefore a probabilistic formulation was used. Further variables are as follows:

- c_{ij} are the costs of the i^{th} resource in the j^{th} alternative,
- q_{ij} is the stochastic variable expressing the capacity of the i^{th} resource in the j^{th} alternative,
- r_{ij} is the stochastic variable expressing the quality of the resource i in the alternative j,
- D is the total demand that should be met with the reliability p_1,
- W is the quality standard that should be secured with the reliability p_2.

Then the formulation of the stochastic program for SMAD is as follows:

$$\sum_{i=1}^{I} \sum_{j=1}^{J} c_{ij} \cdot y_{ij} \rightarrow \text{minimum}$$

Constraints:

$$P \left\{ \sum_{i=1}^{I} \sum_{j=1}^{J} q_{ij} \cdot y_{ij} \geqq D \right\} \geqq p_1$$

for i=1,2,...,I

$$P \left\{ \sum_{i=1}^{I} \sum_{j=1}^{J} r_{ij} \cdot y_{ij} \leqq W \right\} \geqq p_2$$

$$\sum_{j=1}^{J} y_{ij} = 1 \quad \text{for } i=1,2,\ldots,I$$

$$y_{ij} = \{0,1\} \quad \text{for } i=1,2,\ldots,I \quad j=1,2,\ldots,J$$

The stochastic formulation is more realistic and enables the solution even in cases when the deterministic does not exist, however, the program is not linear.

APPENDIX B: LINEAR PROGRAMMING MODEL OF THE RIVER DYJE WATER RESOURCE SYSTEM

The objective function of the model was the maximization of the cost-benefit increase as compared to the present state under constraints:

$$\sum a_{ij} \cdot x_j + \sum a_{ik} \cdot x_k = b_i$$

$$\text{for } i=1,2,\ldots,m \qquad j \in J \quad k \in K$$

$$J \cup K = \{1,2,\ldots,n\}$$

Instead of the equality sign the inequality signs were used in some constraints.

x_j are the continuous variables that model the activities of the investigated water resources system or of the agriculture-irrigation system.

x_k are the discrete bivalent variables that attain the value either 0 or 1. These values are used for decision of yes-no questions or for modelling of uncontinuous or non-linear relations.

b_i are constants in the constraints (or parameters).

J is the set of indices of the continuous variables.

K is the set of indices of the bivalent variables.

Some models of this type can also be described by the theory of graphs and flows in networks. For instance, a discrete optimizing algorithm can be used where x_j and x_k are discrete variables, x_j is expressed in such units that their precision is adequate (mostly 1% precision is sufficient) and the variables x_k are limited to two values. For nonlinearities and yes-no questions, the multigraph can be used.

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OPERATION OF THE LARGEST RESERVOIR SYSTEM IN THE NETHERLANDS: THE IJSSEL LAKE

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1 Introduction

An overview of the main waterway network in the Netherlands and the position of the IJssel lake is presented in Figure 1. The lake has been formed by an enclosure from the sea by means of a 30 km long dike. It is for most part a fairly shallow lake, which had an original surface of 3600 km^2. After several reclamation projects the present lake consists of two parts called the Small IJssel lake (1200 km^2) and the Marker lake (600 km^2) separated by a dike. The two parts are connected through locks. Originally it was planned to empolder the Marker lake, however, reevaluations are taking place and alternative plans are considered, e.g. to reserve this lake for an improved water supply.

The main input to the lake consists of Rhine water. This is brought to the lake by the IJssel river. Upon entering the Netherlands the Rhine divides a first time into the Waal and the Pannerdens channel. The last divides then again in the Lower Rhine and the IJssel river. The flow in the IJssel can be controlled to some extent by a movable weir on the Lower Rhine. Water can be spilled through gates in the closure dike during periods of low tide. Water intake and drainage points are distributed around the lake. For some of the intake and drainage points reliance is made on gravity while in other cases pumps are used.

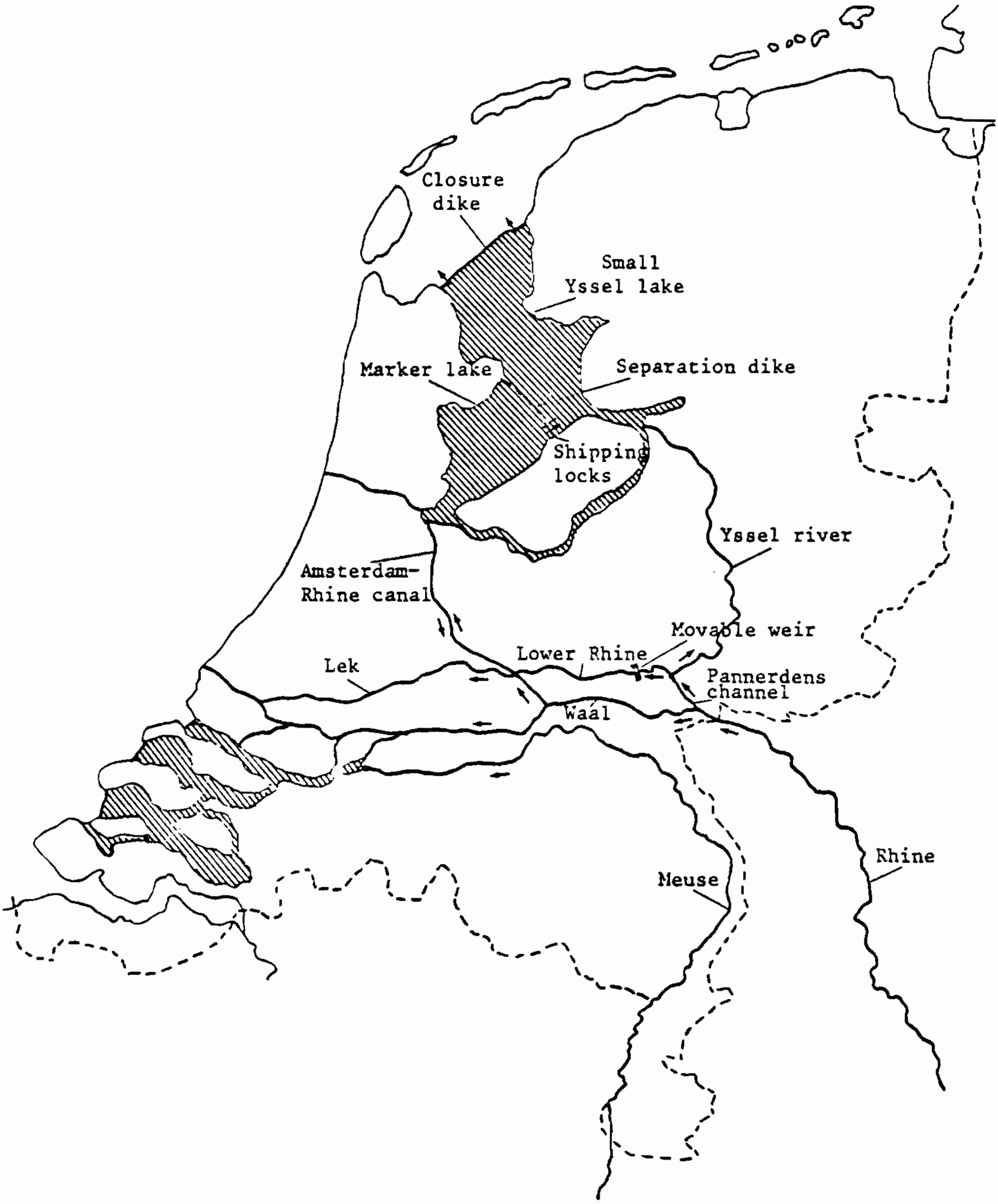

Figure 1 : Position of the Yssel lake system in the waterway infrastructure of the Netherlands

The IJssel lake plays since its formation in 1932 an increasingly important role in the management of water resources in the Netherlands. The initial objectives for the creation of the lake were to increase safety against flooding, to enhance drainage of the surrounding areas, to facilitate land reclamation and to provide a fresh-water buffer for water supply. With increasing demands put on water resources the function of the lake as a fresh-water supply has grown in importance and the conflict between the requirement for a low level in the lake for safety and drainage versus a higher level for a larger fresh-water buffer, has become more pronounced.

The pollution of the Rhine of which the concentration varies considerably over the year has further increased the concern about the buffer capacity of the lake. Up to now the IJssel lake plays a major role in the water resources management of the bordering northern areas of the Netherlands, however, plans are being considered to provide transport of lake water to the southern regions in drier periods. This further increases the importance of the buffer capacity.

A major user of lake water is agriculture, use of water is made for level control, irrigation and flushing of canals and polders in order to combat salinity. The lake serves also as a source of drinking water for some areas. Navigation and recreation are also important.

2 Reservoir operation aspects

Due to the large area of the lake a strong wind set-up of the water surface can occur together with a considerable wave action. Therefore an important aspect of operation is safety. Potential dangerous situations occur when large flows in the IJssel river are combined with unfavourable strong winds and high water levels at sea which limit the possibility for spilling.
In the present operation strategy a particular target level is set for the winter and summer period. The winter target level, 20 cm below the summer target, is set from 1 October until 1 April corresponding to the major storm season. This, however, corresponds also to the period of high water surplus and a water quality, which is considerably better than in the low-flow summer period.

This operation strategy worked fine throughout earlier years and there was actually no need to evaluate the limits to how far one could go with exploiting the buffer capacity of the lake. However, with the increasing demands put on the lake a situation is created where such evaluation seems necessary. In the spring and summer of 1976, which were both dry, a sufficient buffer capacity could not be created and a shortage was felt, while also water quality reached a low point. There have been dry years before, but at that time demands were much lower. Both a higher summer level and an earlier filling of the reservoir should be investigated. A higher water level, however, has considerable consequences on safety, drainage of the surrounding areas and harbour facilities.

Concerning safety, a dynamic policy on target levels could be considered in which a continuous attempt is made to create a sufficient buffer capacity within the limits of safety. A careful evaluation should therefore be made of potential dangerous situations and their frequency of occurrence, the response of the lake and the possibilities for control. A basic element in the operation policy is then the use of forecast information. Both long-term and short-term information is potentially available for the main input source namely the Rhine. Long-term information is available in the form of snow melt predictions in the Upper Rhine basin. Snow melt contributes quite significantly to the flows in the spring and in the summer. The lead time in the prediction can be in the order of several months. The Rhine basin has also a fairly large natural storage capacity and associated base flow which leads e.g. to relatively high correlation coefficients between monthly flows. In the short term large storms in the upper and middle reaches of the basin can cause significant flood waves in the river which have a propagation time to the downstream end in the order of days.
The three types of information should be integrated in the dynamic operation policy with the purpose of obtaining a sufficient buffer capacity within the limits of safety.
Predictions on water quality should also be made and integrated with quantity predictions to obtain the best possible water quality in the lake. The most important quality parameter is salinity. Water quality in the Rhine does follow a seasonal pattern, lower concentrations in winter than in summer, which could indicate a dependency on the flow in the river. However, up to now no significant relationships have been found between quality and

quantity during e.g. the summer season, which could be used to derive quality predictions from quantity predictions. Information on quality will thus mainly be based on a statistical analysis of water quality in the past, no real time information on water quality will probably be available.
Of special interest concerning water quality is the operation of the present two parts of the lake, to obtain the best possible water quality in the Marker lake for water supply. For this purpose the Marker lake has to be considered in series with the small IJssel lake. The interaction between the two lakes is based on gravity flow through the gates in the separation dike. The direction and capacity of this flow depends on the levels in the two lakes and is strongly influenced by wind.

An increase of the lake level beyond present levels influences not only safety but also introduces costs associated with extra measures for adaptation of harbours and to ensure a sufficient drainage by means of new pumping stations and reevaluations of existing stations because of changing pump heads and capacities. These costs should be traded against the benefits of a larger buffer and better water quality. An upper bound on the lake level should follow from such analysis.

The need for a fresh-water buffer is only seasonal and due to the large surplus of water in winter no overyear storage has to be considered.
A major portion of the water will be used for level control, supplemental irregation and for flushing of the polders. These requirements are very much dependent on the actual rainfall input in the growing season, which is quite variable from year to year. As such the demand for water is very much uncertain.

Another factor of uncertainty in the operation of the lake, associated with the large lake area and sensitivity for wind set-up, is the measurement of a representative water level in the reservoir at a particular point in time. Due to its large area a few centimeters mean already a lot of storage.

3 Alternative structural developments

Besides a dynamic operation policy based on the use of forecast information, and the present control possibilities using the spilling gates in the closure dike and the movable weir in the Lower Rhine, some structural alternatives can be considered to improve the operation of the lake and/or the distribution of its water.

- Canalisation of the IJssel river with a set of weirs and shipping locks would make a complete control possible of the flows entering the lake. The present movable weir has only a limited influence on the flows through the back water effect in the Lower Rhine and is mainly operated to improve navigation on the IJssel river. Canalisation would make navigation interests independent of the river flows. It would also create the possibility of completely closing out heavily contaminated Rhine water caused e.g. by an accident.

- The ability to control dangerous high water levels due to a large input of water and/or due to a limited possibility to spill can be improved by considering a large pumping station to complement the spilling gates. Its capacity would be function of the desired level in the lake and the costs have to be traded together with other costs against the benefits of a larger storage.

- Plans are considered to extend the fresh-water supply function of the lake to the southern part of the country by means of the so-called north-south coupling. The Amsterdam-Rhine canal, which is now mainly used for shipping, plays an instrumental role in this scheme. After the necessary infrastructure works the canal would be used for water transport in two directions. In early spring additional water would be sent to the lake to improve storage. In summer when water demand is larger than the supply in the main rivers plus rainfall, water from storage would be sent back to the demand areas using the same canal.

4 Present and future research on the operation of the lake

In the above an overview has been given of the main aspects involved in the day-to-day management of an increased storage. A trade-off has also been suggested between the benefits and the costs of maintaining a larger storage. Before a specific operation strategy can be worked out the role of such storage in the national water management has to be worked out. This should provide a framework with specific objectives and boundaries in which operation of the lake can take place.

A few years back some work on the operation of the lake and its function in the national water budget was done by considering an optimalisation of water allocation over the network of the main waterways in the Netherlands (Fig. 2). For this purpose some demand points were considered on the network and preliminary loss functions were estimated. Initially only quantity was considered then followed by quantity and quality. In those exercises a strong emphasis was put on an identification of suitable techniques to analyse the problems. These initial exercises are being followed presently by an extensive study at the national level of water resources and demands for water in the Netherlands. This study is performed by the Ministry of Public Works together with the RAND corporation (USA) and Delft Hydraulics Laboratory.
The study will provide a national framework for use of water resources. Specifically concerning the IJssel lake it will provide an indication of which structural alternatives are preferred. The study is, however, too broad to be able to cover the real time management of the lake in all its aspects. Plans are therefore being prepared to study the detailed day-to-day operation as soon as results become available. Those are expected in the fall of 1979. Running ahead of the results in the general study some specific aspects of a changing management strategy (e.g. consequences of an increase in the lake level) for the IJssel lake are already being studied by the management agency responsible for the operation of the lake.

In the real time operation strategy, optimization can be used to look at optimal patterns of building up and maintaining a sufficient storage with an optimal water quality, for a forecasted pattern of inflows. However, simulation will probably be needed to evaluate particular aspects of the problem

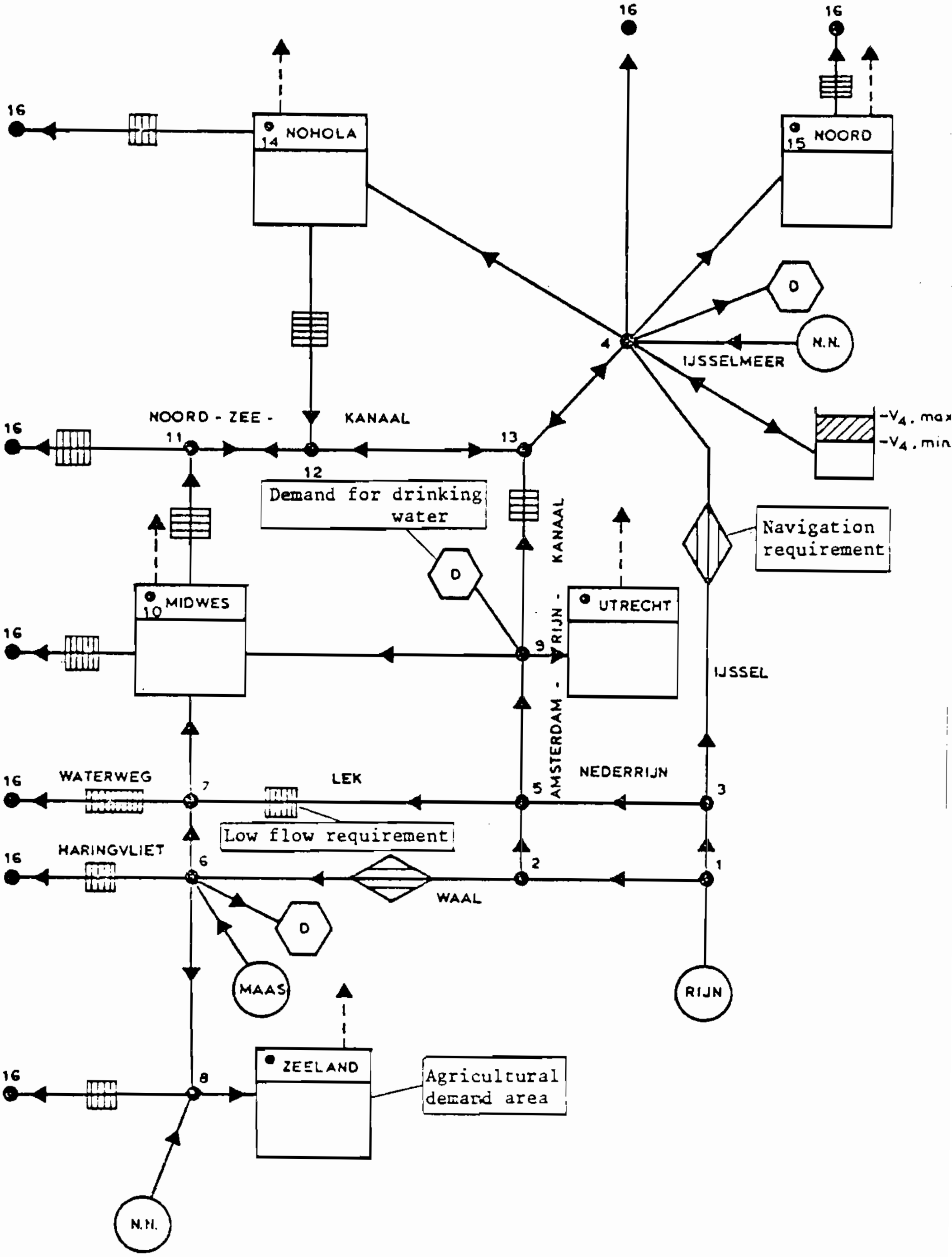

Figure 2 : Schematisation of the main waterway network and demand areas in the Netherlands used in some preliminary exercises on optimal water allocation

such as e.g. wind influence. Such analysis could then provide information e.g. constraints, for another round of optimization.
Different hierarchical levels of operation using information at different time intervals might be considered. Based on long-term information a long-term operation strategy could be determined using large time intervals. The target levels set at these time intervals could be used as boundary condition for a short-term operation using e.g. short-term storm information. As soon as more long-term information becomes available the long-term policy would then be updated.

OVERVIEW ON OPERATION OF MULTI-PURPOSE, MULTI-RESERVOIR SYSTEMS IN JAPAN

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Overview of Reservoirs in Japan

In Japan there are more than 1500 dams (of which about 550 are 20 meters high or more), and some 300 more dams are in the planning stage. Of the existing dams, approximately 660 are situated in what are designated as river areas, and the rest are small ones located outside the river areas, used mostly for irrigation purposes. About half of the 660 major dams are exclusively used for hydropower generation. There are nearly 200 reservoirs which serve multiple purposes. Flood control is one of the prime objectives for most of these reservoirs; other purposes include hydropower generation, irrigation, public water supply, industrial uses and low flow augmentation.

Public reservoir works under the jurisdiction of the Ministry of Construction are categorized into three classes. The first class is called multi-purpose reservoir and is implemented by the Ministry of Construction which is the managing body of major river basins, in keeping with the regulation passed in 1957. The basic idea behind the regulation was to centralize the management of major multi-purpose reservoirs for more effective implementation and operation. The second class of reservoir systems is implemented within the framework of comprehensive planning for designated areas, where industrial development and urbanization require more effective utilization of water resources. The Public Corporation for Water Resources Development was established in 1962 for this purpose. The third class of reservoirs is implemented as a joint venture between water users and the local authority.

Characteristics of Reservoir Operation Problems in Japan

The operation of multiple reservoir systems in Japan provides a slightly different picture than those in other developed countries, because of the geographic, geologic and meteorologic characteristics of the country. Japan is a small and very mountainous country with about three-quarters of the land covered by hills and mountains. Naturally, rivers are short and tend to be rapid-flowing (see Figure 1).

Heavy frontal storms in June and July, typhoons during the August-October period and heavy snows in some areas make Japan one of the regions in the world with the highest precipitation. Mean annual precipitation amounts to about 1800 mm, while the world average is about 800 mm. Also, the precipitation patterns

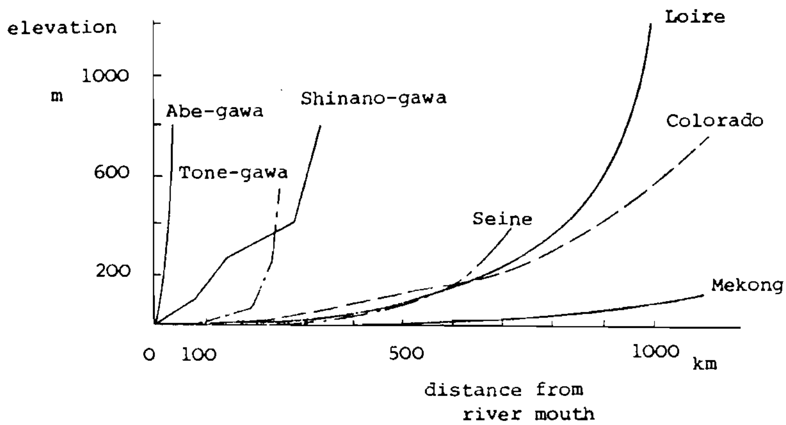


Figure 1. Profile of selected rivers in the world.

are highly variable both in space and time. A hydrograph of flood flow is extremely sharp when compared to rivers in other countries, and the ratio of maximum to minimum flow is one order of magnitude larger for many rivers in Japan (Figure 2).

These characteristics have significant implications for water resources management in Japan. Representative time-series of stream flow (historical or synthetic) that can be used in implicit stochastic models for design and operation of reservoirs are less applicable to many cases in Japan. Thus, more emphasis has been placed on developing rainfall-runoff relationships for

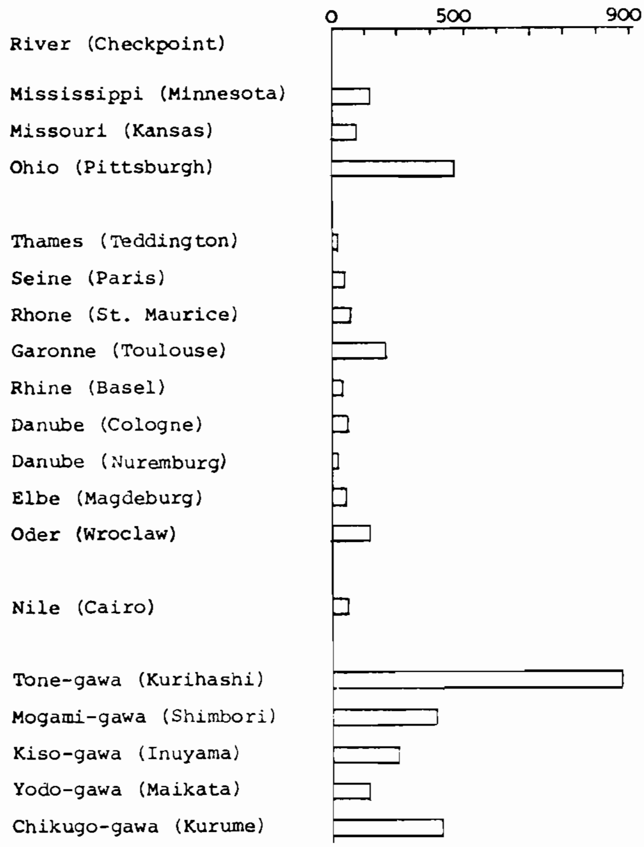
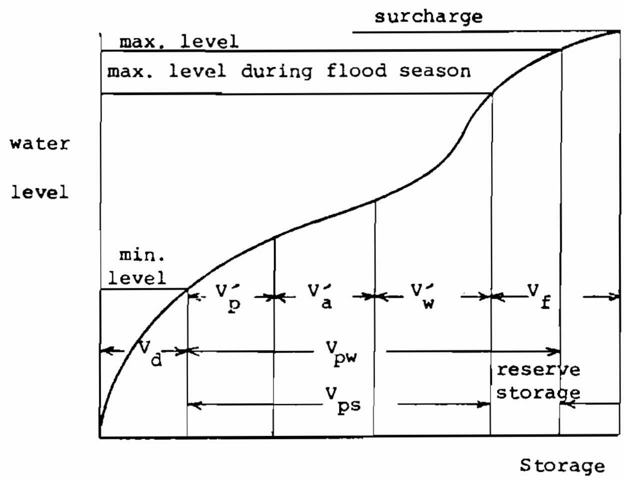


Figure 2. Maximum flow/minimum flow ratio.

particular watersheds rather than on elaboration of operating rules based on "design" streamflow conditions, as will be discussed later in this paper.

Design of a Single Multi-purpose Reservoir

Design of a reservoir largely follows conventional practice (Hanayame and Fuse 1977; Ministry of Construction 1978). A typical design for a multi-purpose reservoir is illustrated by Figure 3. A storage curve for a particular reservoir site is derived and used to determine the capacity of the reservoir necessary to serve different purposes. That is, the requirement for each purpose is specified in terms of water level and/or storage, which are related to each other by the storage curve.



V'_a, V'_w, V'_p	: storage exclusively allocated for irrigation, water supply and power generation, respectively, during flood season
V_{ps}, V_{pw}	: storage for hydropower generation during flood and non-flood seasons, respectively
V_f	: flood storage
V_d	: dead storage

Figure 3. Allocation of storage for a multi-purpose reservoir.

Flood control storage is determined in the following way. First, hydrographs of design floods are determined, based on historical hyetograph and rainfall-runoff analyses. Once the total amount of stormwater to be regulated is specified, the necessary storage capacity is determined, depending on the release policy of the reservoir. Several basic release policies are adopted for many reservoirs in Japan as illustrated in Figure 4. The linear release is considered effective for controlling minor floods with relatively small storage. The variable release policy has been adopted for an increasing number of reservoirs, especially in major river basins. The policy varies the release, depending on the flow at some downstream check points, to attain more efficient utilization of flood storage and more effective peak reduction.

The water supply storage is determined by the cumulative differences between the necessary flow and low flow at the control points. The low flow used in the design is the lowest flow which occurred in ten years, or its equivalent. In Japan, the

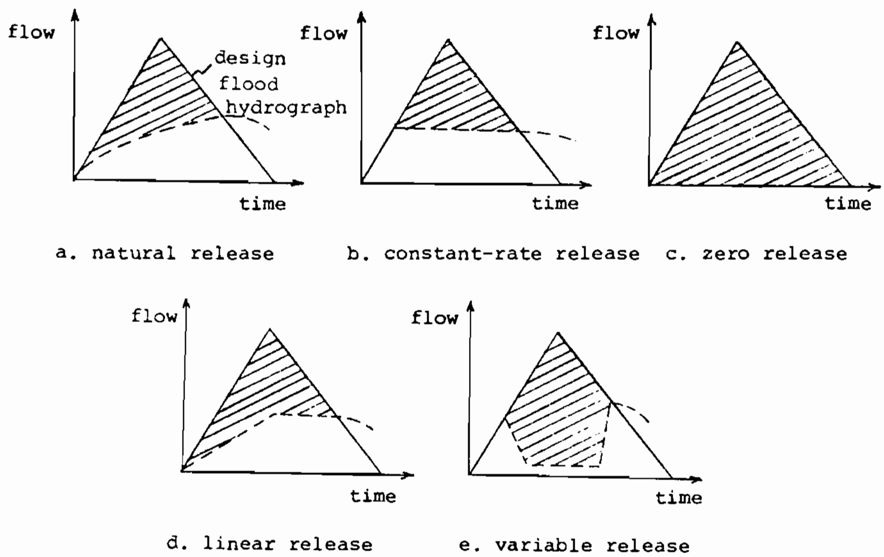


Figure 4. Basic release rules for flood control and flood control capacity (indicated by shaded area).

low flow usually occurs in summer and in winter. In the case of a multi-purpose reservoir, the capacity of the reservoir is usually determined by the storage required in the summer period, since the irrigation period (May-September) and the flood period (June-October) coincide with this low flow period.

Operation Planning of a Multi-purpose, Multi-reservoir System

The operation of multi-reservoir systems in Japan may be described according to the following general guidelines used in planning:

1. Delineate desirable states of the system or provide guidelines for control.
2. Develop operating rules.
3. Structure the process of modifying the operation based on real-time observations.

The first phase in planning the operation of a multi-reservoir system is to specify the uses of the system or the purposes of the operation. The uses, in general, include flood control, water supply for public, industrial and agricultural uses, hydropower generation, low flow augmentation and water quality control. For each use the evaluation criteria must be specified. In the case of flood control, it has been generally agreed that the effects could practically be evaluated only in economic terms (e.g., damage reduction). However, since it is difficult to use the damage reduction directly as the objective function of reservoir operation, it must be related to physical attributes. Peak flow and duration of high flow are considered appropriate as criteria for flood control. Other criteria proposed for other uses include minimization of unnecessary spills and stabilization of flow downstream. The evaluation problem also involves determination of control points at which the effects of reservoir operation should be evaluated especially in the case of flood control.

Operating rules are specified either descriptively or quantitatively. The latter kind of rules are expressed as formula or with the aid of diagrams. Many "rules of thumb" exist and these are practiced in Japan (Ministry of Construction 1977). These include (i) $\frac{Q}{I} = f(q)$ and (ii) $Q = b(I-a) + c$ for a single reservoir, where Q and I are respectively the outflow and the inflow of the reservoir; q is the flow at the downstream datum point and f is a function; a, b and c are parameters. For multiple reservoirs, one proposed way is to release water from each reservoir in such a way that (iii) $\frac{K_i-S_i}{I_i-Q_i}$ = constant for all the reservoirs, where Q_i and I_i are the outflow and inflow of the ith reservoir, K_i and S_i are the capacity and the remaining storage of the ith reservoir.

Developing operating rules requires characterization of inflow processes to each reservoir, as well as evaluation of the effects of operation on the downstream control points. The former, in turn, involves specification of design storms in the form of hyetographs and development of rainfall-runoff relationships to obtain inflow hydrographs. The rainfall-runoff relationships range from a relatively simple unit hydrograph method to highly sophisticated models including control theoretic models. The storage function method (Linsley 1976; Yashikawa 1972) is most widely used in large river basins. Tank models (Sugawara 1972; Sugawara et al. 1975) are also used in some river basins in Japan.

The operating rules thus developed must be modified to adapt to real situations which cannot be completely forecasted. The modification of the operation in the past, however, has been done more or less in an *ad hoc* manner. More integrated treatment of this phase with other phases described above is desirable. In this area, application of modern control theory (e.g., Kalman filtering) is seen as promising (see, for example, Hino 1977).

Illustration of Multi-reservoir Systems in Japan

The locations of major river basins with multiple reservoirs are indicated by Figure 5. Operation management systems with data collection networks and computers have been established for

1. Ishikari-gawa River Basin
2. Kitakami-gawa " "
3. Tone-gawa " "
4. Kiso-gawa " "
5. Yodo-gawa " "
6. Yoshino-gawa " "
7. Chikugo-gawa " "

Figure 5. Location of major multi-reservoir systems in Japan.

all the major multi-reservoir systems, but no system functions quite satisfactorily, mainly due to difficulty in short- and long-range forecasting of rainfall. Brief descriptions of real-time operation of two major multi-reservoir systems in Japan are given below.

Upper Tone-gawa River Basin

The upper Tone-gawa river basin consists of several tributaries and covers an area of 5114 km^2, north of Tokyo's metropolitan area (see Figure 6). There are five reservoirs--Yagisawa, Fujiwara, Aimata, Sonohara and Shimokubo--located on the different tributaries. The storage and the release capacity of these reservoirs are given in Table 1. The system has served several purposes including flood control, water supply for public, industrial and irrigation uses and hydropower generation. The main emphasis, however, in determining operating rules of the system has been placed on flood control. Effects on other uses have been considered only on an incidental basis.

Basic rules have been determined for each reservoir, based on its release and storage capacity and effects on downstream control points for design flood. Yagisawa is located together with Fujiwara, the furthest upstream; Fujiwara and Aimata have limited release capacity and are less adaptable to frequent changes in release. Thus the constant rate release policy is used for these reservoirs. Sonohara has small storage capacity as compared with its large drainage area. Because of the difficulty in forecasting runoffs for this area, the release policy is defined conditional on the expected magnitude of flood. Shimokubo is operated by the variable release policy, because of its dominant effects on flood control in the upper Tone-gawa river basin.

A typical real-time operation of the system may be sketched as follows. Yagisawa, Fujiwara and Aimata reservoirs are operated according to the following predetermined basic rules. For Sonohara and Shimokubo reservoirs, the points A and B in Figure 6 are taken as control points. Depending on the expected hydrograph

Figure 6. The upper Tone-gawa river basin.

Table 1. Capacity of reservoirs in upper Tone-gawa river basin.

Reservoir	Yagisawa	Fujiwara	Aimata	Sonohara	Shimokubo
Flood Control capacity [Mm^3]	22.1	21.2	9.4	14.1	35.0
Release capacity [m^3/s]	380	28	10	1190	970
Drainage area [km^2]	167.4	227.2	110.8	492.2	322.9

of flood, the peak reduction necessary at these points is determined and the amount of permissible release from each reservoir is determined by backtracking the flow. The storage function method is used to simulate the flow along the river. The operation is modified by changing the parameters of the release rules on observing differences between computed and actual hydrographs.

<u>Yodo-gawa River Basin</u>

The Yodo-gawa river basin, located in the Osaka metropolitan area, has a drainage area of about 7281 km^2. The basin consists of three main tributaries--Katsura-gawa, Uji-gawa and Kizu-gawa, and Lake Biwa, the largest lake in Japan (Figure 7). Four reservoirs--Takayama, Shorenju, Murou and Amagase, with a capacity of 56.8, 27.2, 16.9, and 26.3 million cubic meters respectively, and Setagawa weir--are located on the tributaries. The uses served by the system include flood control, water supply, hydropower, flow regulation and water quality control.

A typical mode of operation of a single reservoir for flow regulation (especially low flow augmentation in the July through October period) is illustrated by Figure 8. A Tank model has been calibrated and used to simulate the runoffs. Use of dynamic programs to derive joint operating rules has also been suggested, but is not in operation yet.

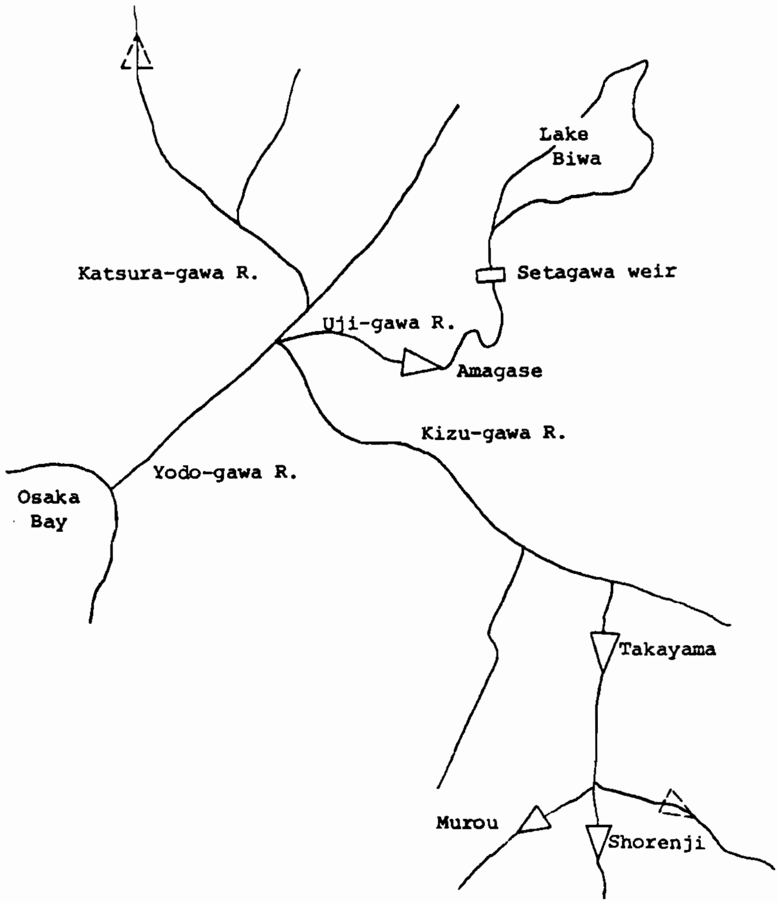


Figure 7. The Yodo-gawa river basin.

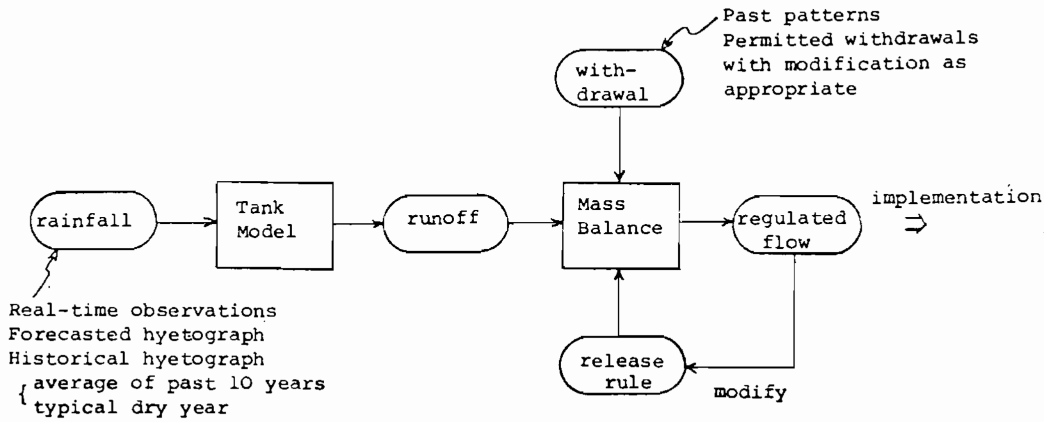


Figure 8. Scheme of real-time operation of a reservoir in the Yodo-gawa river basin.

The flood control operation in this system is similar to that in the upper Tone-gawa river basin described above. A control point is specified for each reservoir and the basic release rule as shown in Figure 9 is determined by simulation based on the storage function method so that the peak reduction is maximized at the control point.

Conclusion

Although Japan is categorized as a high-precipitation region, average per capita precipitation is only one-fifth of the world average. Short flow times due to high gradient river beds and high regional and seasonal variation of flow make the situation in Japan even less favorable.

More effective use of water resources are naturally of utmost importance. Of all the reservoirs at the planning stage, over 60% are for multiple purposes. Development of more effective operating rules by using sophisticated techniques, however, is not considered most important for a more efficient utilization of multi-purpose, multi-reservoir systems. Instead, much emphasis has been placed on development or rearrangement of more appropriate institutional frameworks. Problems involved in

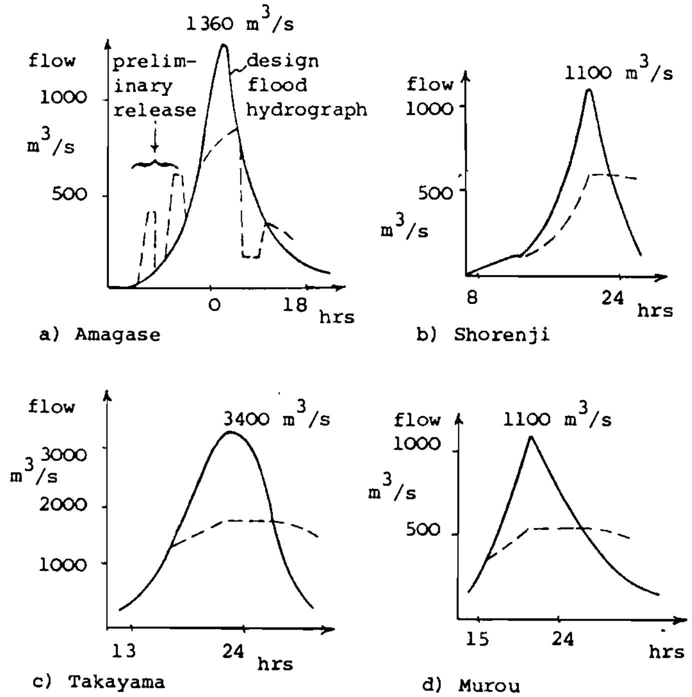


Figure 9. Basic flood control release rules for reservoirs in the Yodo-gawa river basin.

reservoir operation, which would call for appropriate institutional arrangements involve the following: conservation of water, rationing during shortage, conjunctive management of water quantity and quality, flexible use and substitution of water supply storage for flood control, compensation for negative effects on hydropower generation due to flood control release and resolution of upstream/downstream and other types of conflicts.

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MULTIRESERVOIR MANAGEMENT OF THE TRENT, SEVERN, RIDEAU AND CATARAQUI SYSTEMS: A CASE STUDY

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INTRODUCTION

The Trent, Severn, Rideau and Cataraqui systems are located in Ontario, Canada (see Figure 1). Collectively, they form part of a historic and recreational waterway from the St. Lawrence and Ottawa rivers to Lake Ontario and on to Georgian Bay in Lake Huron. Overall responsibility for the operation and maintenance of this waterway and for controlling releases from most of the reservoirs resides with Parks Canada, a branch of the Canadian Department of Indian Affairs and Northern Development.

Although the four watersheds are not overly large, they are very complex systems to operate. The Trent basin for example has 92 reservoirs and numerous unregulated lakes. The basin contains a portion of the Trent-Severn navigational waterway as well as 14 hydro plants. Other water-based considerations include flood control, water-based recreation, water supply, water quality and fish and wildlife preservation.

Through the years, there have been increasing problems in operating the various multireservoir systems to satisfy the different water-based needs. The problems have arisen because of growing population pressures with associated increases in water-based demands. The principal conflicts include.

(a) Excessive reservoir drawdown during summer recreational periods (especially during hot, dry summers) to satisfy downstream water quality and navigation requirements.

(b) Inequitable relationships between levels on different reservoirs, especially during summer recreational periods.

(c) Inadequate flood control storage (both during the late winter-spring period and during the summer recreational period) while ensuring that levels are sufficiently high to satisfy navigation and recreation needs.

(d) Excessive level fluctuations spawning and hatching periods (especially for lake trout) while ensuring that seasonal storage needs are being satisfied.

(e) Nonoptimal scheduling of reservoir releases for downstream hydro requirements while ensuring that recreational, navigational and other water-based needs are being satisfied.

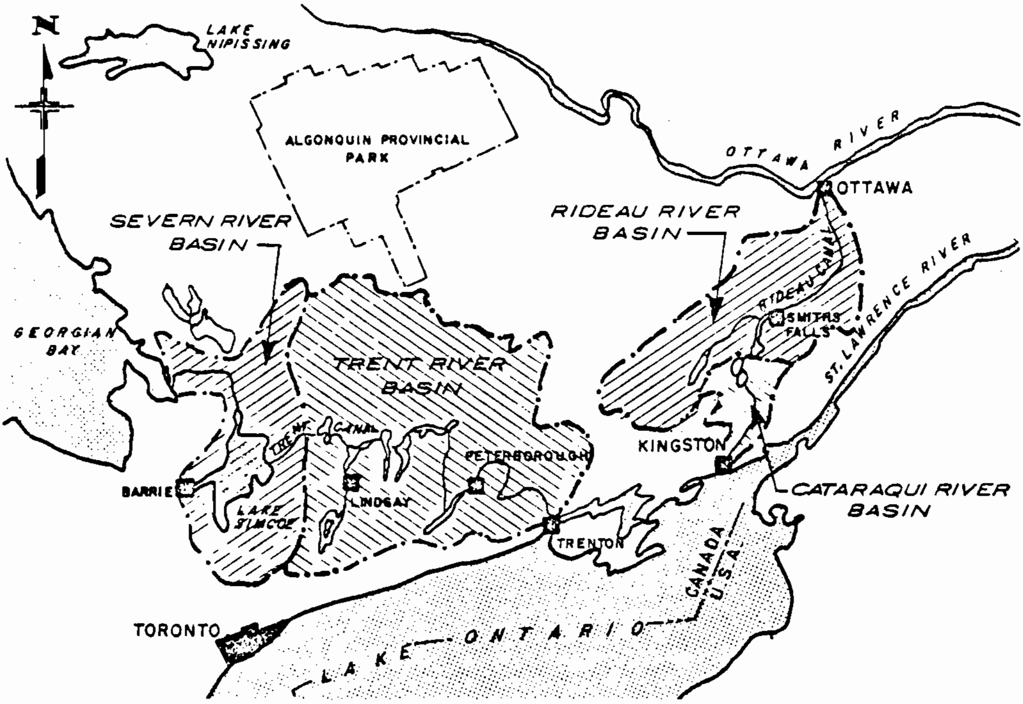


FIGURE 1 - LOCATION OF TRENT, SEVERN, RIDEAU AND CATARAQUI RIVER BASINS

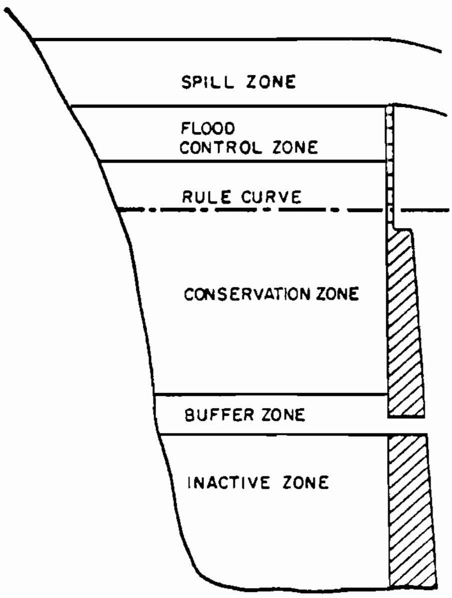


FIGURE 2 - FRAMEWORK FOR REPRESENTING INDIVIDUAL RESERVOIR

In 1972, Acres Consulting Services was commissioned to undertake an assignment for a study on operational procedures in the Trent watershed. The basic purpose of this assignment was to review existing operational practices, to explore different operational strategies and to provide appropriate recommendations which would lead to optimal operation of the system.

For performing this initial assignment, Acres developed a general multipurpose, multireservoir model.[1] This model permitted simulations to be carried out for a wide range of system-wide operating strategies.

After completing the initial assignment, the mathematical model was modified so that it could be used as a dispatch tool, i.e., to aid in determining reservoir releases on a day-by-day or week-by-week basis. The model has been used in this manner since 1974.

In a subsequent study on the Severn basin, the principal emphasis was on exploring the benefits of various physical works to improve satisfaction of system-wide water-based needs.

In a further investigation on operational practices in the Rideau and Cataraqui watersheds (the study was performed simultaneously for these two basins), the emphasis was on formalizing and improving system-wide operating procedures (similar, in principle, to the earlier investigation on the Trent basin). In this study, it was recognized that substantial improvements could be effected by implementing a formalized forecasting and operating procedure for the late winter-spring period.[2] The proposed procedure was implemented in 1977 and has been used successfully for the past three years.

The purpose of this paper is to describe the basic principles and procedures used by Acres to examine and improve the operation of multireservoir systems. The first section contains a summary of alternative modeling strategies and describes the basis for selecting a simulation model for studying operational practices. In the next section, some basic concepts of operational planning are discussed. This is followed by a summary description of the Acres multipurpose, multireservoir model. The application of the model to the Rideau basin is then summarized. The last section contains concluding remarks.

ALTERNATIVE MODELING STRATEGIES

In order to explore basin-wide benefits for different operating strategies, it was recognized that any of several different modeling strategies could be used. These were examined in detail and are summarized elsewhere.[1,3] The basic options were as follows

- use of explicit stochastic optimization models
- use of implicit stochastic optimization models
- use of decision rules
- trial-and-error procedure using simulation models.

With explicit stochastic optimization models, reservoir releases are derived by including probability distributions of inflows directly in deriving optimal releases policies. Some of the early investigative work with this approach was carried out by Thomas and Watermeyer[4,5], Dietrich and Loucks[6], Loucks[7], Gablinger and Loucks[8], Bather[9], and Falkson[10]. These investigations were based on the use of linear programming, dynamic programming or a combination of these two methods. In initial investigations, it was assumed that inflows had known probability distributions, but were independent random events. In later investigations, serial correlation effects of sequential inflows were included.

The explicit stochastic optimization approach is the only approach, in principle, which leads to truly optimal solutions--aside from problems of discretization. In practice however, this approach is very expensive. For example, Gablinger and Loucks[8] showed that a single solution using LP resulted in 2,000 equations and 15,000 variables, and required two hours of computer time (IBM 360/65 computer). The only known application to a multireservoir system was performed by Schweig and Cole[11]. They applied dynamic programming to a two-reservoir system and found that computational costs were high, even with very simplified inflow representations.

With the implicit stochastic optimization approach, releases are optimized for a stochastic hydrologic sequence. With this approach, it is implicitly assumed that there is perfect foreknowledge of all future hydrologic inflows.

After optimizing reservoir releases for the given sequence, appropriate regression analyses are performed on the simulated results to derive a reservoir release policy. The form of this equation is prescribed by the user and is, of course, open to question.

Early developmental work using the implicit stochastic optimization approach was carried out by Hall[12], Hall and Buras[13] and Young[14,15]. Although this approach is also computationally expensive, it has more promise for multi-reservoir systems--since the size of the problem does not increase as quickly as for the explicit approach.

Since both of the above approaches were generally too expensive and too impractical for most real-life situations, it became obvious that some simplifications were desirable. In 1969, ReVelle, Joeres and Kirby[16] put forth the idea of the linear decision rule. For a reservoir system, they suggested that the reservoir release, q, during a particular time period could be related to the storage at the start of the time period, s, by the relationship

$$q = s - b$$

where b is a decision parameter to be derived by the model. This rule had the decided advantage that it could be translated conveniently and efficiently into LP formulation.

Since its introduction however, the linear decision rule has been subject to considerable controversy. Revelle and Kirby[17], Joeres, Liebman and Revelle[18], Nayak and Arora[19,20], Eastman and ReVelle[21] and Leclerc and Marks[22] have modified, extended and/or applied this method to reservoir management problems. However, Eisel[23], Loucks[24], Sobel[25], and Loucks and Dorfman[26] have all questioned the utility of this model for reservoir management. For example, Loucks and Dorfman[26] have demonstrated that the use of the decision rule leads to conservative results, primarily because the imposition of the rule itself represents an additional operating constraint. They suggest that this technique may be suitable for screening studies, but seriously question its utility for deriving optimal operating policies.

For the foregoing, it can be readily appreciated that optimization models have major limitations for deriving multireservoir operating policies--especially systems with many reservoirs. In order to use these models effectively, it is often necessary to simplify the system representation, to limit the length of the hydrologic sequence and to eliminate many detailed considerations which occur with operating multireservoir systems in practice.

In many systems, it is also difficult (if not virtually impossible) to define the objective in mathematical terms. While defining economic benefit functions for some water-based needs (hydropower and irrigation, for example) is reasonably well established, this is not the case for systems which produce certain other water-based benefits--recreation, fish and wildlife preservation, etc.

For the studies on the Trent, Severn, Rideau and Cataraqui systems, it was considered desirable to reflect operating procedures in detail. There were also several water-based benefits which were judged to be very difficult to quantify in economic terms. These included recreation, fish and wildlife, water quality and recreational navigation. For these reasons, it was decided that it would be best to use a simulation model and to adopt a trial-and-error approach in developing optimal system-wide operating strategies.

It is useful to elaborate on one additional principle of the Acres approach. At the outset, it was considered most desirable to develop a simulation model which would effectively simulate recent historical operating practice. This included not only the simulation of the physics of the system, but also simulating the various decision and monitoring processes for various states of the system for different hydrologic conditions.

This process of calibrating recent historical operating practices served two purposes. Firstly, it provided a basis for formalizing rules of system operation for recent and current operating conditions. And secondly, it provided a basis for exploring the potential basin-wide impact associated with modifying current rules of system operation.

This process had a very positive side benefit. For the initial calibration work, it was necessary to have very detailed discussions with system operating staff. From these discussions, it was possible to define rules of system operation in sufficient detail so that the simulated results were in reasonable agreement with historical operating responses. It was not possible, of course, to achieve perfect calibration since some historical procedures have an "ad hoc" element. Nevertheless, after a series of discussions and model tests, it was possible to achieve sufficiently close agreement so that both the consultant and the system operating staff were satisfied that the model provided a reasonably accurate representation of system operating rules.

This process provided a unique opportunity for system operating staff to actively participate in the study. In many cases, the operating staff played a major role in defining potential improvements in the rules of system operation--as well as in the interpretation of the simulated responses.

This collaborative effort during the study permitted rapid acceptance of the consultant's recommendations. Indeed, all the major recommendations of the various studies on the Trent, Severn, Rideau and Cataraqui systems were implemented very quickly.

REPRESENTATION OF SYSTEM OPERATION

Before describing the simulation model, it is useful to discuss concepts and principles of system operation.

Concept of Operation

At the outset, it was necessary to develop a basic perception of the manner in which a multireservoir system is operated. It was recognized that there are two basic activities.

(a) Decision Making - At certain intervals of time, the chief operator will review the condition of the system and make a set of operating decisions. The decisions are expressed in release rates at the various control structures.

(b) Monitoring - During individual time intervals, the operating staff monitor the system in accordance with the prescribed operating decisions.

The operating decisions are based on three considerations

- the current state of the system, i.e., existing reservoir levels and channel flows
- forecast of net inflows
- the operating policy.

The operating policy is a set of rules which define the best decision procedure for any combination of system state and inflow forecast. This may either be prescribed precisely, i.e., formalized or may be a set of ideas which the chief operator and his staff have developed from years of operating experience.

The simulation model was developed to simulate these decision and monitoring processes. It proceeds recursively from one-time interval to the next--with each time interval reflecting the interval period between decisions. At the start of each interval, it derives decisions which simulate the operator's decision process. This is based on satisfying the prescribed operating policy optimally for the given system state and inflow forecast.

Framework for Representing Alternative Operating Policies

There are four aspects associated with defining rules of system operation

- defining rules for individual reservoir levels
- defining rules for individual channel flows
- defining relationships between reservoir levels
- defining relationships between reservoir levels and channel flows.

These will be described in turn.

Rules for Individual Reservoir Levels

The framework adopted for defining reservoir levels includes both rule curve and multiple zoning representation (see Figures 2 and 3). The rule curve denotes the locus of optimal operating levels over time.

For either wet or dry hydrologic conditions, the levels in the various reservoirs will deviate above or below their respective rule curves. The zoning representation provides a convenient framework to define operational practices to reflect satisfaction or concern for different water-based needs. These include

(a) Conservation Zone - A zone within which the various water-based needs continue to be satisfied. The rule curve, which is always positioned in this zone, is the optimal operating level.

(b) Flood Control Zone - The zone immediately above the conservation zone--used as a storage reserve for abnormally wet periods. Normally, the level of the top of this zone denotes the onset of significant flood damage.

(c) Flood Damage (or spill) Zone - The zone immediately above the flood control zone. In this zone, flood damage is occurring. When levels reach this zone, it is normal to resort to emergency operating procedures.

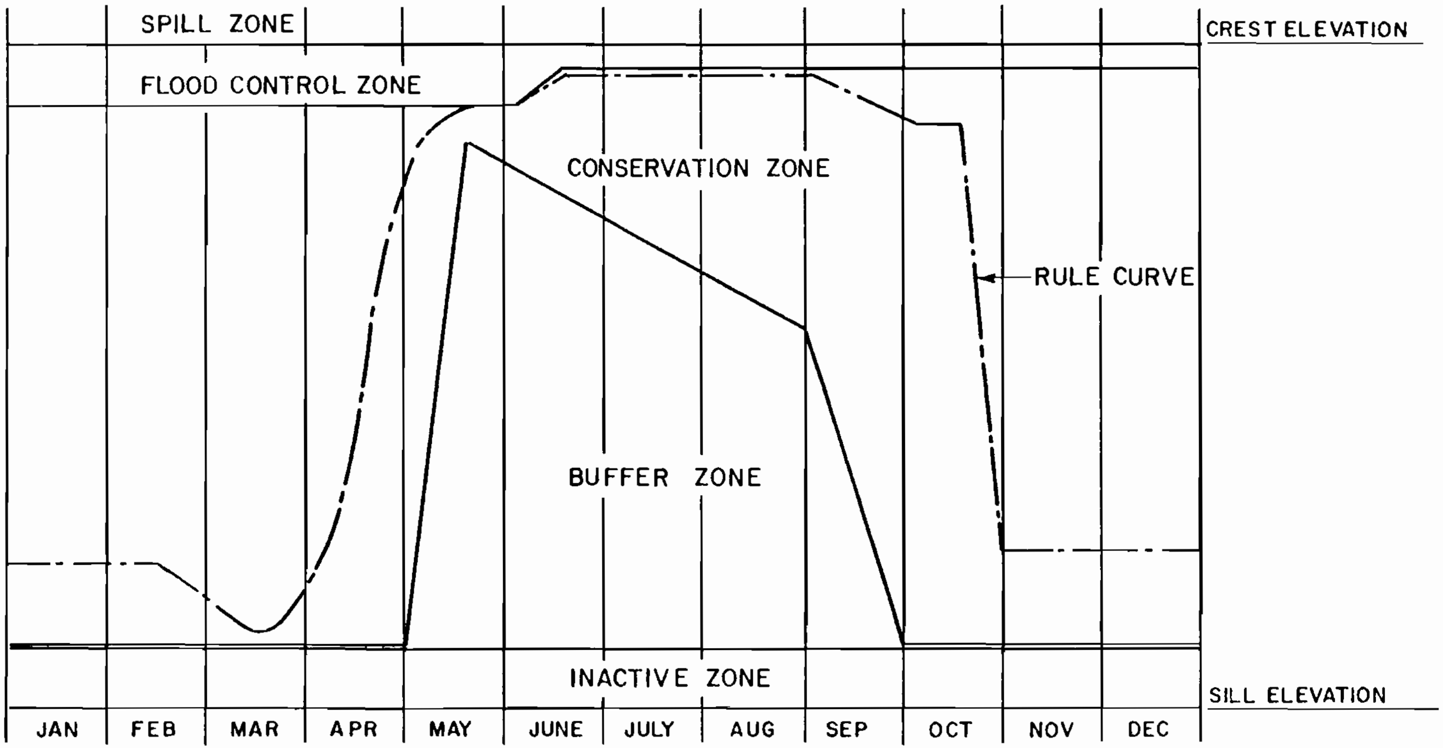


FIGURE 3 – VARIATION OF ZONES AND RULE CURVE FOR A TYPICAL RESERVOIR THROUGH AN ANNUAL CYCLE

(d) Buffer Zone - The zone immediately below the conservation zone--used as a reserve for abnormally dry periods. When levels reach this zone, it may be necessary to reduce outflows so that only the most essential needs are satisfied--domestic and industrial water supply, for example.

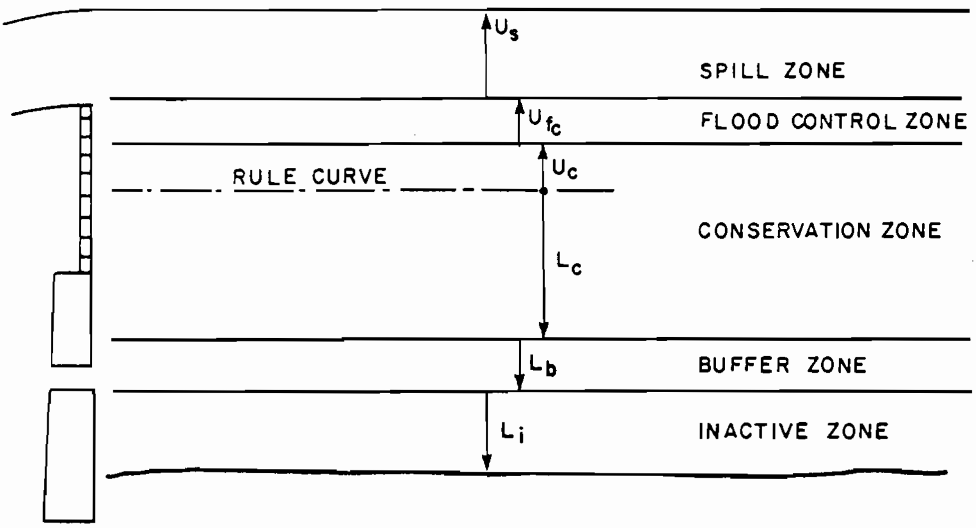
(e) Inactive Zone - The zone immediately below the buffer zone. This zone is often defined as the dead storage zone, i.e., corresponding to levels below the sill of the outlet structure. However, the top of this zone may be positioned at a higher level to satisfy a prescribed institutional or legal requirement--such as minimum navigation level. When reservoir levels reach the inactive zone, it may be necessary to reduce releases to satisfy emergency needs only.

Rules for Individual Channel Flows

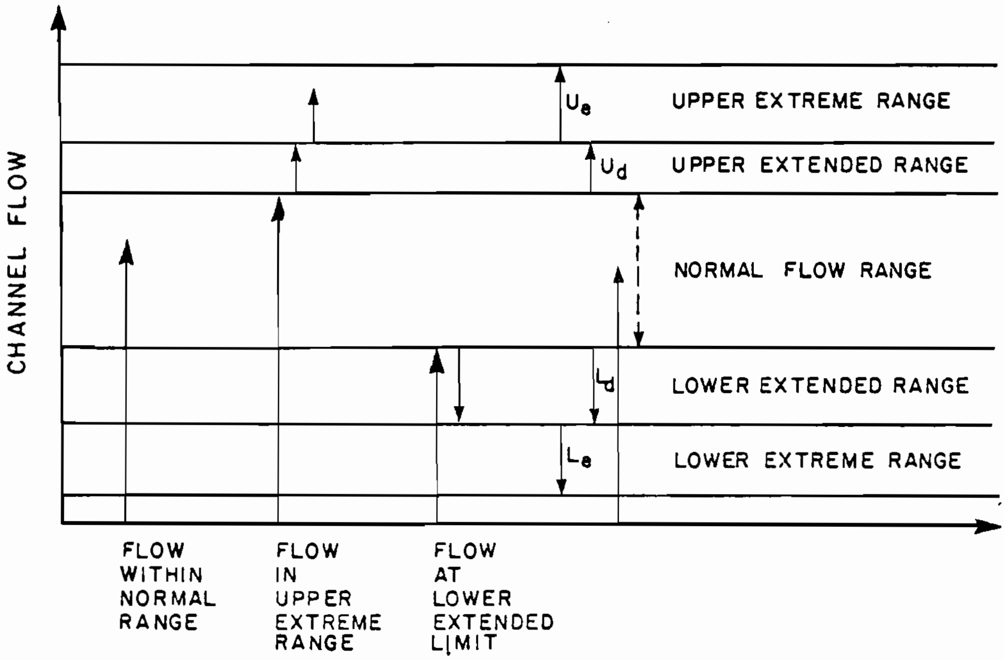
Channel flows were represented in a similar way (see Figure 4). For each channel, it was assumed that there was a desirable flow range within which it was perceived that conditions were more-or-less ideal. If the upstream reservoirs were operating in their respective conservation zones, then the individual channel flows would be maintained within their respective normal flow ranges.

For abnormal conditions, ie., when one or more upstream reservoirs are in either their respective flood storage zone (for wet conditions) or their buffer zone (for dry conditions), it may be desirable to increase the operating range (referred to as the "extended range"). This permits water to be released more quickly during wet periods or to be conserved during dry periods. The extension beyond the normal range normally reflects some inconvenience along the channel--such as local flooding during wet periods or reduction of water supply for certain water-based needs during dry periods.

For extreme (or emergency) conditions, it may be desirable to extend the operating range even further. This is associated with one or more upstream reservoirs being in their flood damage zone (for extreme wet conditons) or in their inactive zone (for extreme dry conditions). The further extension of the flow range may reflect significant damage along the channel reach--such as extensive flooding (wet conditions) or severe curtailment of supply (dry conditions). These conditions are imposed to provide a suitable mechanism for balancing damages around various reservoirs with damages along channel reaches.



RESERVOIR REPRESENTATION



CHANNEL FLOW REPRESENTATION

FIGURE 4 - REPRESENTATION OF COMPONENT ARCS FOR RESERVOIR STORAGE AND CHANNEL FLOW

Interreservoir Balancing

There are two aspects to discuss when considering rules for balancing reservoir levels.

Firstly, it is considered that all reservoirs should be operating in the same zone to the maximum extent possible. For example, it is preferable that a group of reservoirs all be in their respective flood control zone rather than having some in the flood control zone, some in the flood damage zone and some in the conservation zone. The idea of maintaining all reservoirs in the same zone to the maximum extent possible has been implicitly assumed in formulating the simulation model.

Secondly, there are three general classes of policies for defining interreservoir relationships.

- Priority Policies

- "Equal Function" Policies

- Storage Lag Policies

For priority policies, the reservoirs are ranked. The complete zone of the lowest priority reservoir is fully utilized first, the second lowest priority next and so on. It should be noted that the priority rating of the various reservoirs can be iefined differently for each zone.

For equal function policies, the zonal volume of all reservoirs is utilized together, while at the same time, maintaining some interreservoir relationship. This may be expressed as "same percentage of zonal volume", "equal elevation deviation from zonal limit", etc.

For storage lag policies, a subset of reservoirs is allowed to deviate a certain amount before another subset begins to deviate. Further deviation then occurs together while maintaining a prescribed differential between the two subsets. The differential may be expressed as percentage of zonal volume, storage volume, elevation difference, etc.

In the simulation model, these three policies have all been modeled. It is also possible to prescribe a mix of these policies.

Reservoir Level - Channel Flow Relationships

In addition to satisfying prescribed interreservoir relationships, it is also necessary to prescribe relationships between reservoir levels and channel flows. These have been briefly discussed above when describing channel flow representation. It is useful however, to provide two additional comments.

Firstly, even though it is desirable in principle to maintain all reservoirs in the same zone, an operational limit on a channel flow may lead to situations where groups of reservoirs above and below the channel are operating in different zones.

And secondly, it should also be noted that the limits on physical discharge capacity (both upper and lower) may override operational limits. This can also have the effect of creating subsets of reservoirs which are operating in different zones.

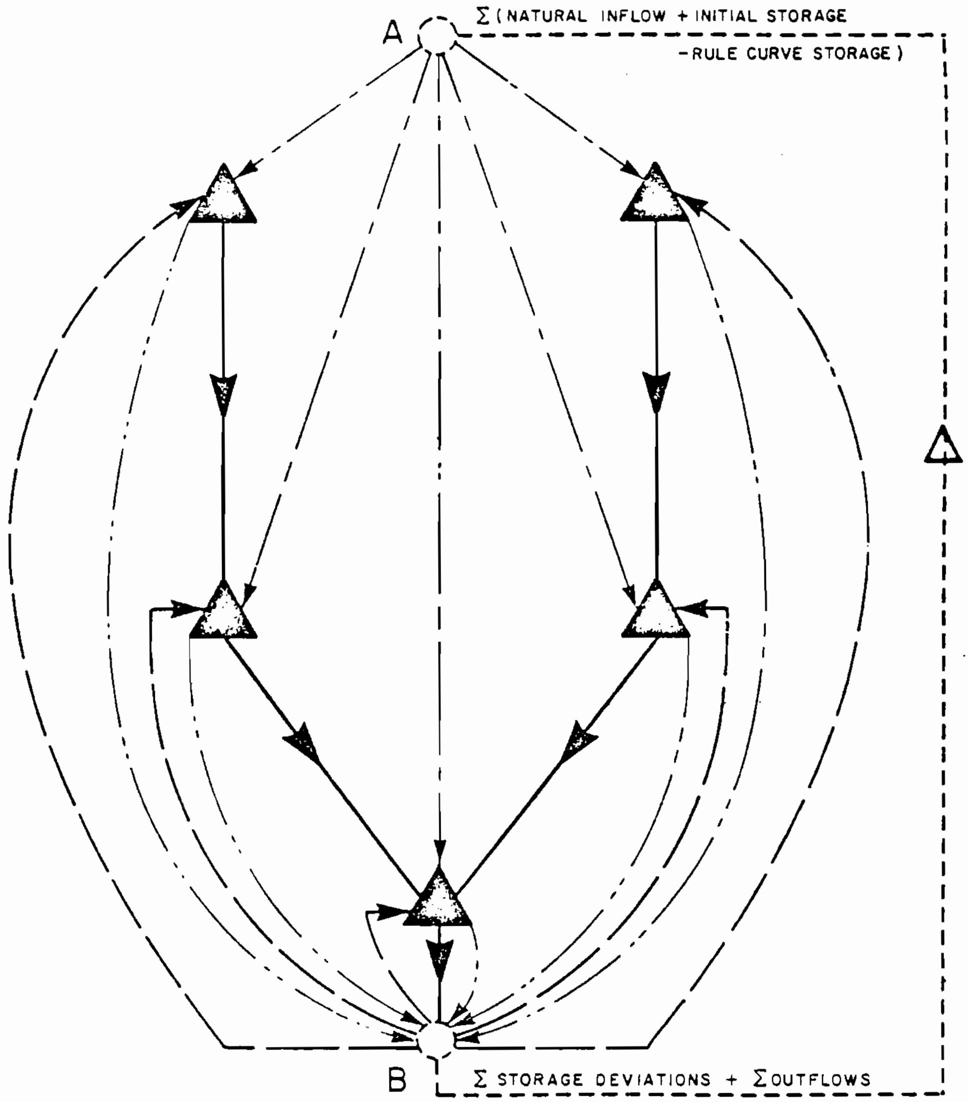
DESCRIPTION OF MATHEMATICAL MODEL

The Acres' multipurpose multireservoir simulation model has been developed to simulate the various aspects of system operation which have been described above. In developing the model, it was also considered most desirable that the model be sufficiently flexible so that any of a variety of operating strategies could be prescribed with input data. It was also necessary that the model be programmed so that any arbitrary configuration of reservoirs and interconnecting channels could be prescribed.

To achieve these and other requirements, the simulation model was developed with the inclusion of the out-of-kilter algorithm[27,28]. The out-of-kilter algorithm is an optimization program which solves any problem which can be represented as a closed network diagram (see Figure 5). The variables of the model are the flows q_{ij} along the individual "ij" arcs

where i denotes start node of arc

j denotes end node of arc.



LEGEND

CHANNEL FLOW ARC - INCLUDES THE 5 ARCS, $N_{ij}^{\dagger}$, $U_{dij}^{\dagger}$, $U_{eij}^{\dagger}$, $L_{dij}^{\dagger}$, $L_{eij}^{\dagger}$ (SEE FIGURE 6)

UPPER STORAGE DEVIATION ARC - INCLUDES THE 3 ARCS, U_c, U_{fc}, U_s (SEE FIGURE 6)

LOWER STORAGE DEVIATION ARC - INCLUDES THE 3 ARCS, $L_c^{\dagger}$, $L_b^{\dagger}$, $L_i^{\dagger}$ (SEE FIGURE 6)

NET SYSTEM INFLOW ARC

FIGURE 5 - RESERVOIR SYSTEM IN CAPACITATED NETWORK FORM

In mathematical form, the out-of-kilter algorithm can be stated as

$$\text{Min } Z = \sum_{ij} c_{ij}\, q_{ij} \tag{1}$$

subject to

$$\sum_i q_{ij} - \sum_i q_{ji} = 0 \qquad \text{for all } j \tag{2}$$

$$L_{ij} \leq q_{ij} \leq U_{ij} \qquad \text{for all } ij \tag{3}$$

where

Z is the objective function

c_{ij} is the cost of each unit of flow q_{ij}

L_{ij} and U_{ij} are the lower and upper bounds respectively on q_{ij}

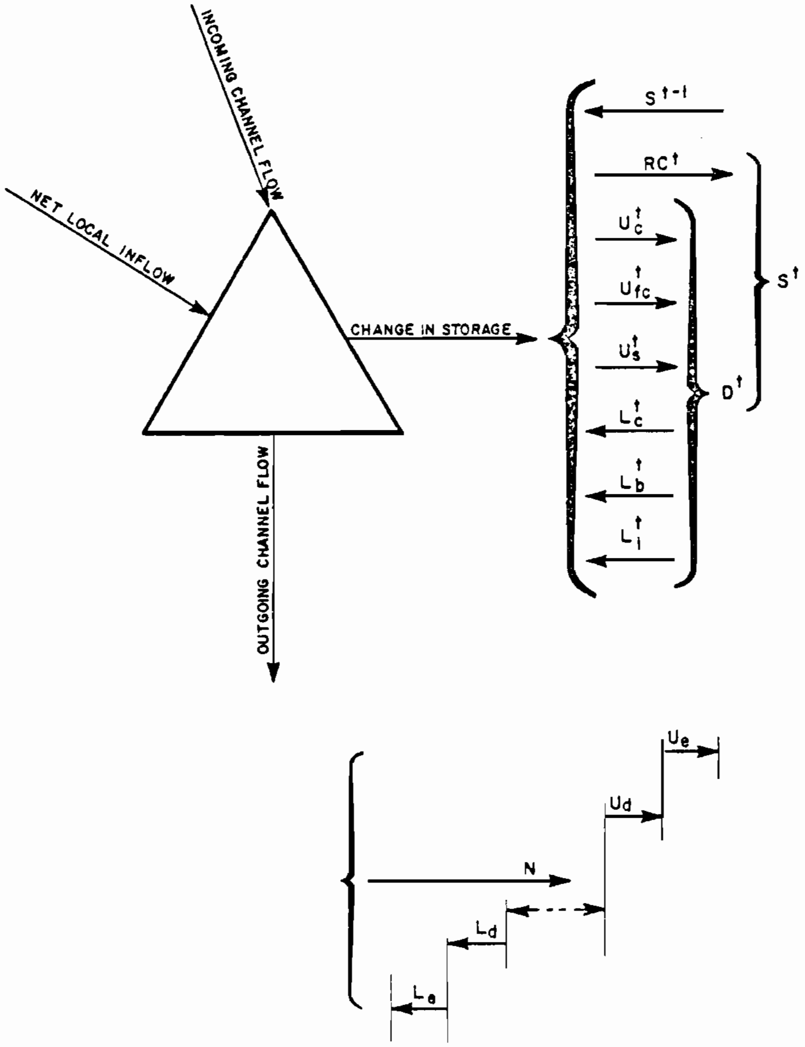
It is a relatively straightforward task to represent any multireservoir schematic in network form--as demonstrated in Figure 5.

In order to represent rules of operation for the individual reservoirs and channels, as prescribed above, it was necessary to define a series of arcs for each reservoir and channel (see Figure 4). One arc is necessary to define the deviation in each zone in each reservoir and one arc to reflect each flow range in each channel. The overall continuity requirement at a storage node is shown schematically in Figure 6 (refer also to Figures 4 and 5).

In order to represent the different policies of operation as prescribed above, it was necessary to define penalties for those variables which denoted deviations from ideal conditions. This includes those arcs which represented deviations from the rule-curve elevations in the various reservoirs and those which represented flow ranges outside the normal flow range. This is displayed on Figure 7.

The cost coefficients in Equation 1 can be used to represent system operating policies. In principle, they can be used as measures of penalty against ideal system operation--therefore, the use of the term "penalty coefficients".

In general, the penalty coefficients increase in value in the following sequence (to simulate operating decisions for progressively wetter conditions)



Q_L	+	ΣQ_{IN}	−	ΣQ_{OUT}	=	ΔS	(UNIT TIME STEP)
LOCAL INFLOW		INCOMING CHANNEL FLOWS		OUTGOING CHANNEL FLOWS		CHANGE IN STORAGE	

FIGURE 6 - RESERVOIR STORAGE AND CHANNEL FLOW ARC REPRESENTATION

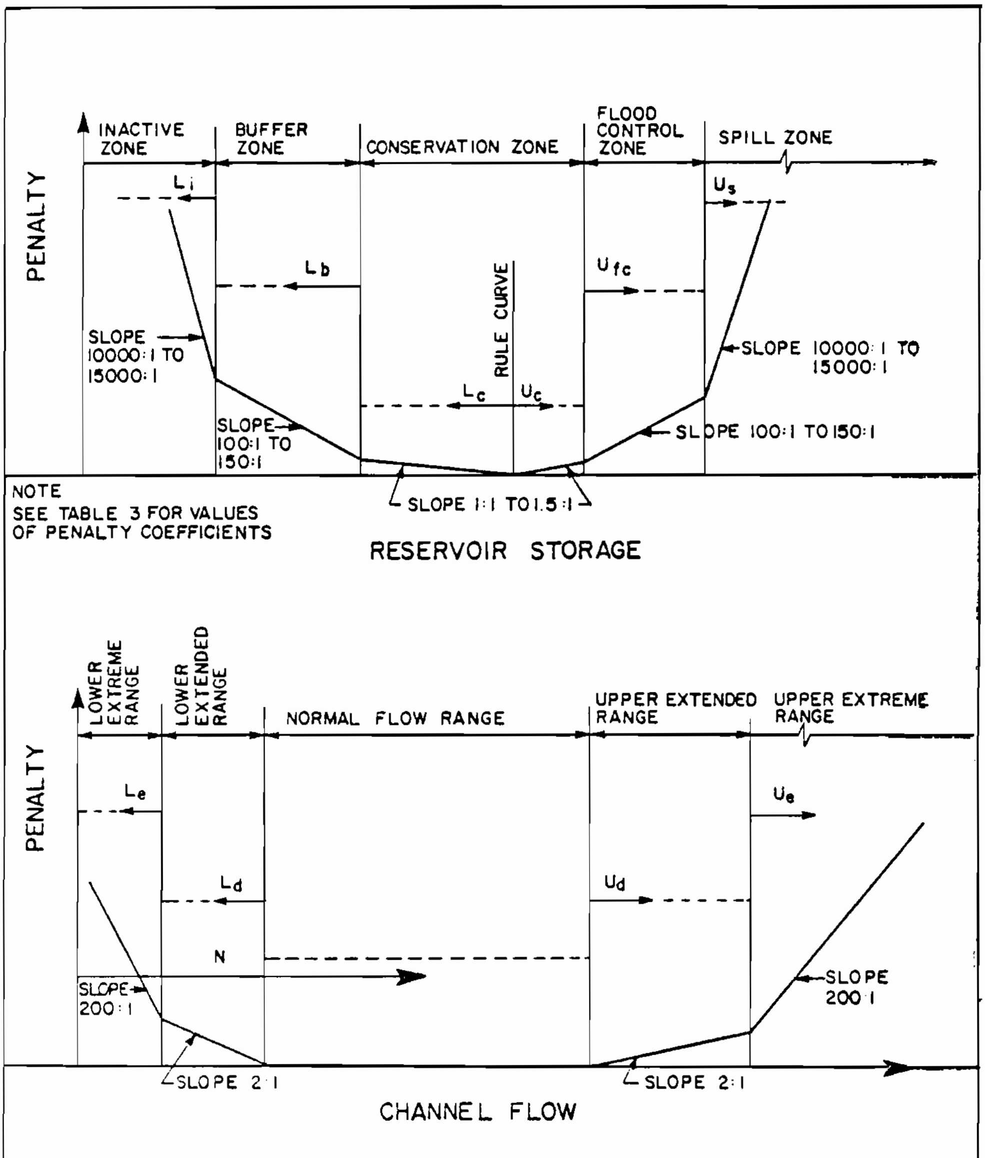


FIGURE 7 - PENALTY REPRESENTATION FOR RESERVOIR STORAGE AND CHANNEL FLOW

- deviation above rule curve in conservation zone
- deviation in extended flow range
- deviation in flood control zone
- deviation in extreme flow range
- deviation in flood damage zone.

This ensures that solutions are obtained which satisfy general relations between reservoir levels and channel flows. This also ensures that all reservoirs are maintained in the same zone to the maximum extent possible.

The same principles apply for reflecting operating decisions for progressively drier periods.

To satisfy prescribed interreservoir relationships, additional considerations are necessary. For priority policies, the reservoirs are ranked. This is achieved by assigning the smallest penalty coefficient to the lowest priority reservoir, a slightly higher value to the second lowest priority reservoir and so on. With this representation, the model will then endeavour to utilize the entire volume of the lowest priority reservoir first, then begin to utilize the zonal volume of the second lowest and so on.

For equal function and storage lag policies, the penalty coefficients are all given idential values. However, with this representation, the out-of-kilter submodel does not produce a unique solution--even though the objective equation (Equation 1) is minimized. It has been necessary, therefore, to develop an additional routine which distributes the deviations between the reservoirs to satisfy the prescribed equal function or storage lag relationship.

EXAMPLE PROBLEM

Results of the most recent study of the Rideau basin have been selected to demonstrate the use of the model for calibrating system operation and to demonstrate its utility for developing optimal operating procedures. These results are described in detail in Acres' report to Parks Canada[29].

A schematic of the system is shown in Figure 8. Basically, the system includes four reservoirs and a lake (Christie Lake) with a constricted outlet. The Rideau Canal follows the Rideau River, passing through Lower Rideau, Big Rideau and Upper Rideau Lakes before crossing the watershed divide into the Cataraqui system (see Figure 1). Bobs Lake and Wolfe Lake are reservoirs which are used to augment downstream flows, especially during dry summers.

The calibration of system operation was performed for three historical years.

1957 - very dry spring followed by below-average summer precipitation

1972 - very wet spring followed by above-average summer precipitation

1975 - average spring followed by below-average summer precipitation

By simulating system operation for these three years, it was judged that the system was being tested for a sufficient range of hydrologic conditions. It was considered, therefore, that the model results would truly reflect the full range of operating conditions.

Results of the final simulation for 1975 are displayed on Figure 9. From these and other results, it was considered that the model had produced a reasonably accurate representation of operating practice. In most cases, the differences between actual and simulated results were less than three inches. The most significant discrepancy occurred on Big Rideau-Lower Rideau Lake during the spring freshet period. After detailed discussions however, it was noted that actual operation during this period had been a departure from historical operating practice and that the model was a more reasonable representation of expected responses if normal operating procedures had been followed.

After completing the calibration of system operation, several changes in operating procedure were proposed and tested with the model. The most significant recommendation was to develop a formulized forecasting and operation procedure for the late winter-spring period[2]. It was proposed that operating decisions be made at half-monthly intervals from the beginning of February to the beginning of May. On the first and fifteenth day of each month, a projection of cumulative runoff from that day to the middle of May was performed. This was based on variables such as the amount of snow cover, cumulative runoff, precipitation, etc. These relationships were derived from 30 years of recorded historical information.

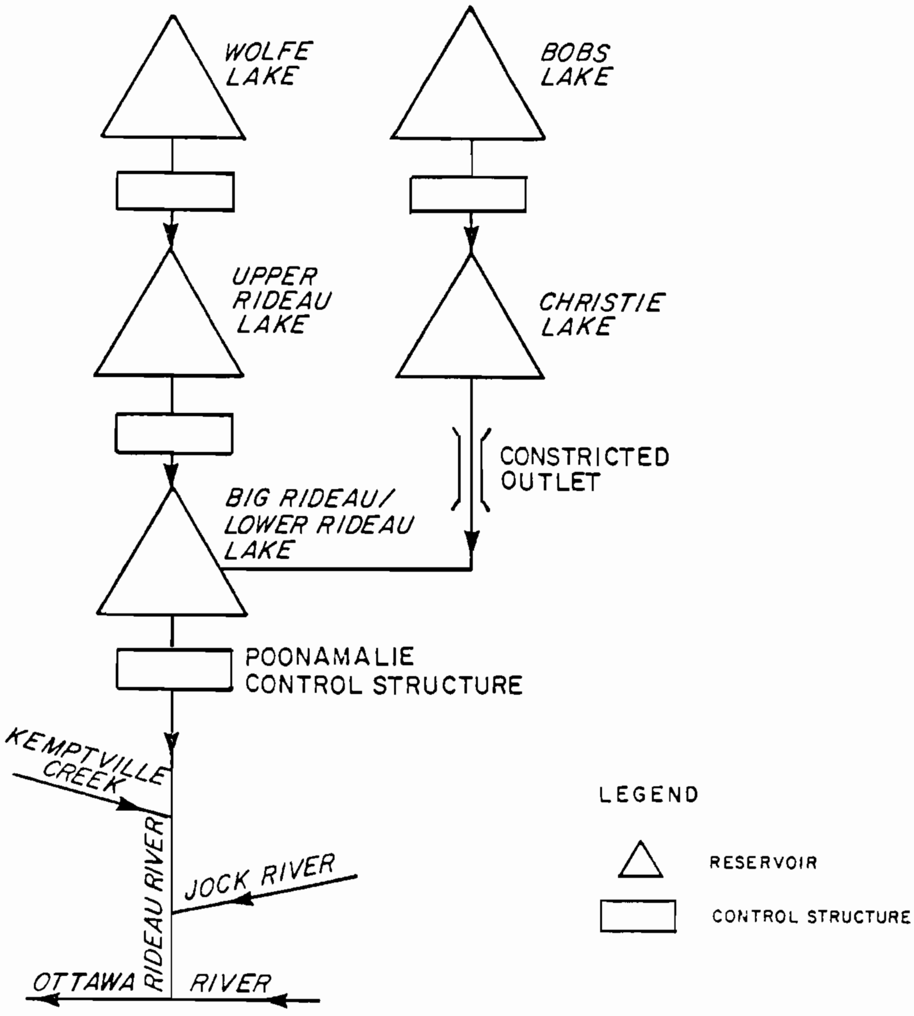


FIGURE 8 - SCHEMATIC OF RIDEAU SYSTEM

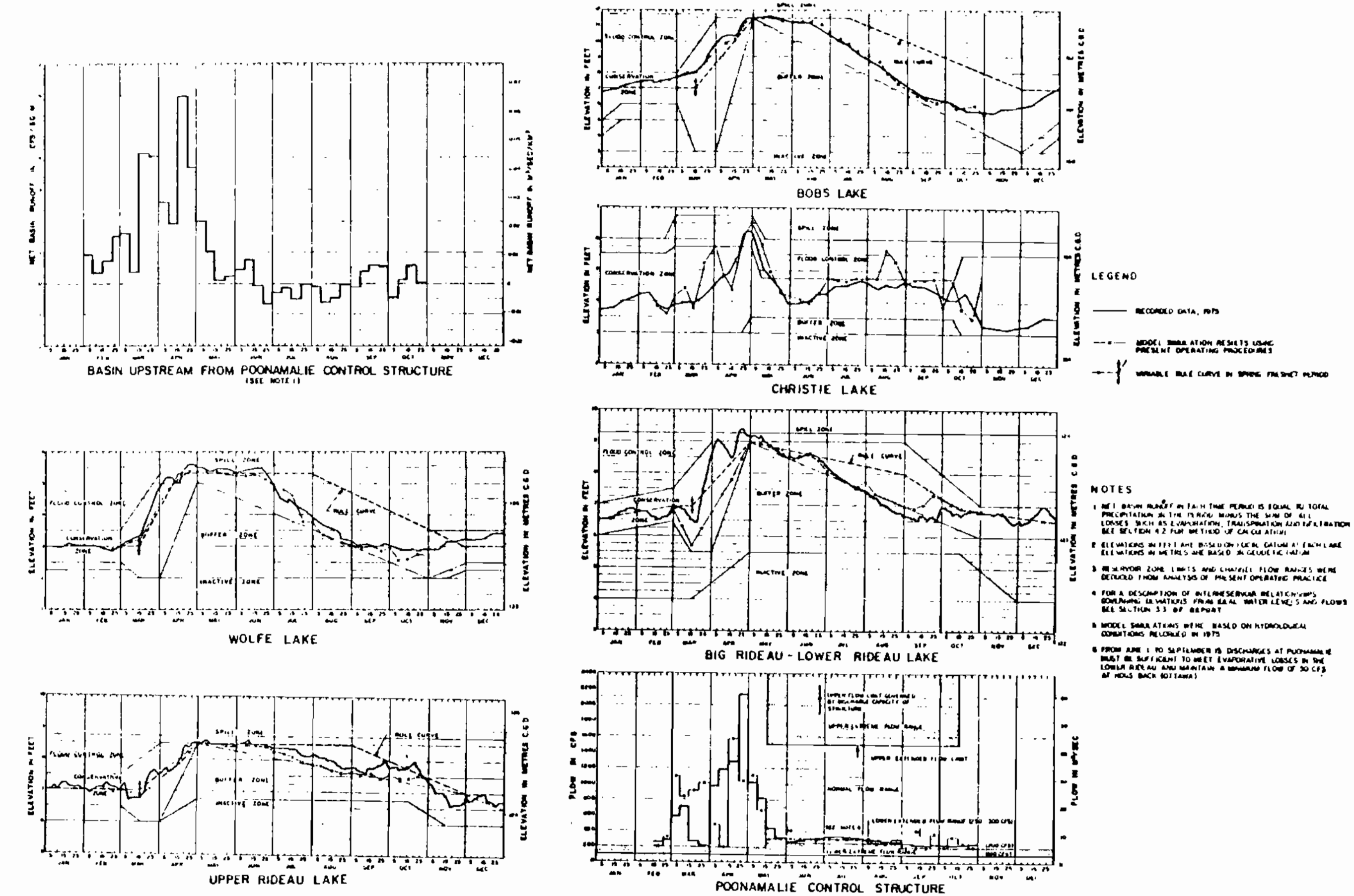


FIGURE 9 – MODEL SIMULATION USING PRESENT OPERATING PROCEDURES - RIDEAU SYSTEM

Appropriate release rates were then computed. These were based on two principles. Firstly, it was presumed that it would be most desirable to maintain a more-or-less uniform release rate at each control structure for the remainder of the late winter-spring period. Allowance was made to reduce flows to an absolute minimum during the peak of the freshet period--to minimize contribution to potential flooding downstream, especially around the City of Ottawa.

And secondly, special care was taken to ensure that the various reservoirs reach their May 15 target levels--to minimize the risk of low levels during the summer recreation period. Accordingly, the targe releases were checked (and in some cases, modified) to ensure that the reservoirs were not drawn too far.

This procedure was tested for the three years (1957, 1972 and 1975). From the simulated responses, it was noted that there were distinct improvements in system response in both wet and dry years. In wet years, the simulated responses reflected larger withdrawal from the various reservoirs and later filling than for historical conditions. This provided confirmation that the procedure reduced risk of flooding in years of high runoff.

In years of low runoff, the simulated results showed earlier filling than for historical conditions. Again, all the reservoirs come much closer to meeting their May 15 target levels than occurred historically.

There were also several other modifications which were tested with the model. The flood control zone was expanded in the early summer period to reduce incidence and magnitude of flood damage, rule curves and zonal boundaries were modified, wintertime holding levels were adjusted and flow ranges were altered (especially for the winter period).

During and after submitting the report, Acres worked closely with the Rideau Canal Authority in implementing the principal recommendations of the study. For implementing late winter-spring forecasting and operating procedure, the consultant worked very closely with the Authority for two years. Results were excellent, even with runoff being well above average in both years. Downstream flows during the peak of the freshet period were also reduced substantially. Similarly, flooding around the various reservoirs and along the channel reaches were reduced.

CONCLUDING REMARKS

In the Trent, Severn, Rideau and Cataraqui basins, the Acres' multipurpose multireservoir simulation model has proven to be a very valuable tool for testing various strategies of watershed management. It has provided a reliable and credible tool for calibrating recent historical operating practice and for testing potential improvements in rules of system operation.

The use of the model has also provided a very valuable basis for interacting with system operating staff. In calibrating the operation of the system, it has been necessary to ensure that the staff are closely involved in describing the rules of system operation and in interpreting the simulated responses. After completing the calibration exercise, the staff are again involved in defining potential improvements in operating rules and in interpreting simulated responses. This has provided a good basis for implementing the proposed recommendations and for giving full support to modern approaches in watershed management--including the use of computerized dispatch (as for the Trent basin).

Since completing these assignments, Acres have used their model on other operational management investigations, both in Canada and abroad. These include operational control for the Volta River in Ghana and for the Chao Phraya and Mae Klong rivers in Thailand. The model has also been used for planning studies--the South Saskatchewan River in Alberta, Canada, the Salmon River in Newfoundland, Canada and the Karun River in Khuzestan, Iran. This model has, therefore, become a basic tool for river basin planning, project feasibility assessment, operational planning and system dispatch.

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OPERATION OF MULTIPLE RESERVOIR SYSTEMS: A CASE STUDY OF THE UPPER VISTULA SYSTEM (An Introduction)

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1. INTRODUCTION

The purpose of this paper is to illustrate through the example one of the existing multireservoir systems in Poland, how reservoir operation rules can be developed with the application of some theoretical tools. This will take into account the decision making and information structures and their patterns. Therefore, attention is focused not only on methodological developments, but also on the kind of information necessary for the determination of operational rules, and their informational structure.

First, the system considered is described and the objectives of water management are specified. Next, two methods for determining the parameters of operation rules are presented, and finally the hierarchical approach to reservoir control in the system considered is described.

2. A BRIEF DESCRIPTION OF THE UPPER VISTULA SYSTEM

A general layout of the Upper Vistula system is shown in Figure 1. The system includes five storage reservoirs. The Goczalkowice reservoir (1) is located on the Upper Vistula River,

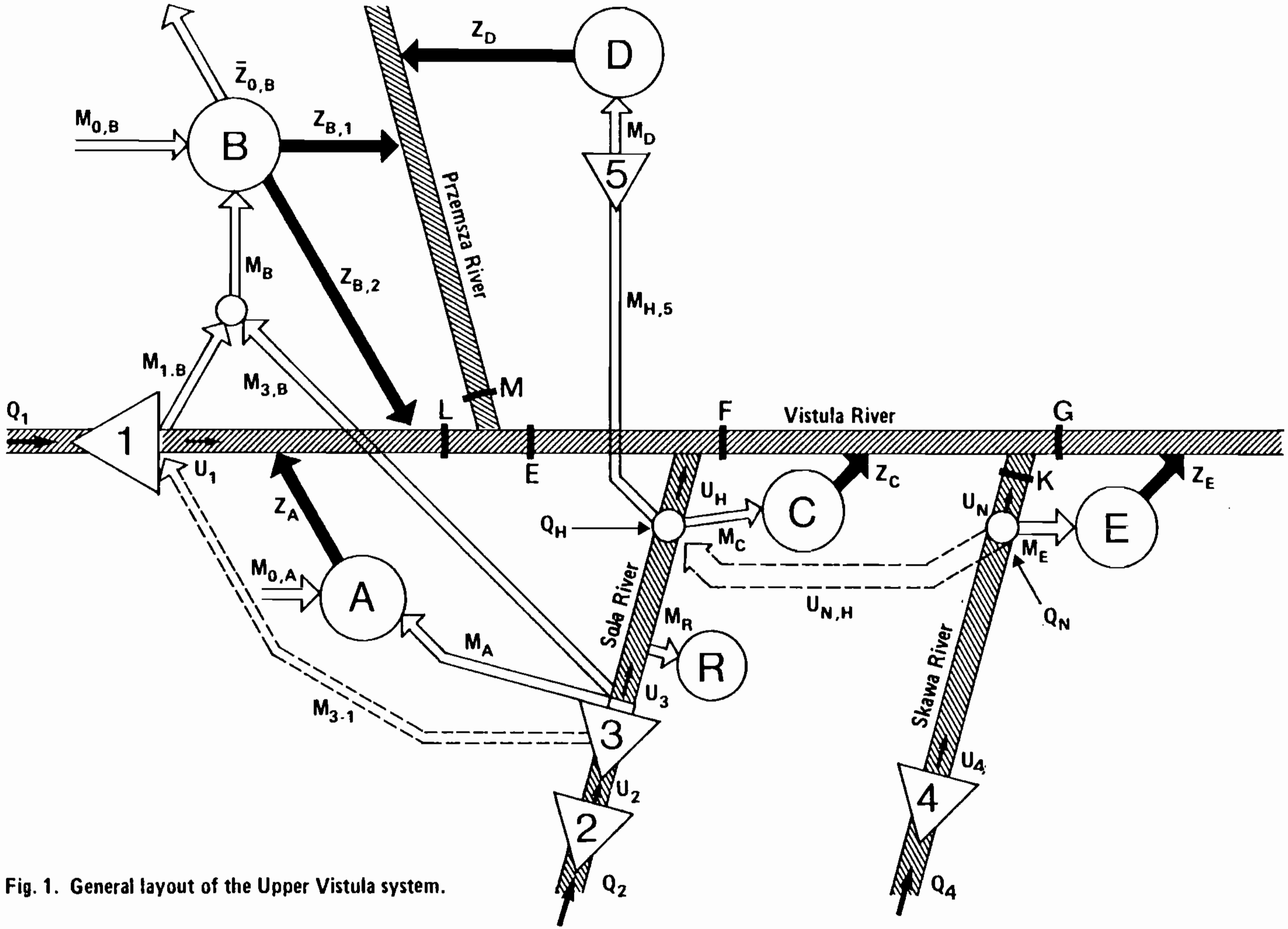

Fig. 1. General layout of the Upper Vistula system.

while the Tresna (2) and Porabka (3) reservoirs are located on the Sola River. Immediately below Porabka is the small Czaniec reservoir which is not shown in Figure 1. The system can be easily expanded by the Swinna Poreba reservoir (4) to be located on the Skawa River. A decision concerning construction of this reservoir has already been made by the authorities concerned. Finally, there is the off-the-river Dzieckowice reservoir (5), which was built as a buffer (compensation) reservoir for one of the major water users in the area.

The major objectives of the Upper Vistula system are to secure water supply for the industrial and municipal water users referred to in Figure 1 as A, B, E; to supply the steel works D with water from the Sola reservoir via the Dzieckowice reservoir, and to supply water to chemical plant C, and fish farms R. At the same time, concentration of several pollutants, which are mainly discharged into the Vistula River downstream of the outlet of the Przemsza River, should be maintained at the levels compatible with water quality requirements. Reservoirs (2) and (3) are provided with hydroelectric power stations; however, this study focuses on water supply and water quality considerations. The flood control portions of storage capacities are not taken into account.

The principles of water resources management in the Upper Vistula and Sola river basins have not changed for a long time, with the Goczalkowice reservoir being operated independently from the Sola reservoirs and vice versa. The Goczalkowice reservoir is operated for the constant release rate, and it supplies water mostly to user B. The Sola reservoirs equalize streamflows in that river, mostly for the purpose of supplying water to users A, B, and C. Water requirements of fish farms denoted in Figure 1 as "R", are not taken into account by the present operation rules of the Sola reservoirs. Decisions concerning the fulfillment of these requirements are made when the need arises; they are mostly based on the actual streamflow rate at the outlet of the Sola River. The operation of water transfer facilities from the Sola River to the Dzieckowice reservoir is under the control of user D. The transfer rates are decided upon by user C, depending mostly on the actual storage level of the Dzieckowice reservoir. Quite often the transfer rates are decided upon by "intervention," depending on user C's requirements. Although there are a few general rules to follow, in principle, operational decisions concerning Sola reservoirs are based on ad hoc agreements among all parties concerned. What is important, however, is that water quality requirements of the Vistula River, downstream of the Sola outlet, are not accounted for in the operational decisions concerning releases of water from the Sola reservoirs.

To summarize, storage reservoirs in the Upper Vistula system are not operated at present as a system of interrelated flow control facilities. It is felt that efficiency of the system performance could be considerably enhanced if the interdependencies among the reservoirs (and among some of the users) are more explicitly taken into account in the operation rules developed for the system as a whole.

The general layout of the complete system is shown in Figure 1, but for modelling purposes, only some selected elements of the system and interactions among these elements are considered.

3. THE MONTE CARLO APPROACH TO OPTIMIZATION OF THE OPERATION RULES

During the past few years, the first attempt to develop operation rules for the Upper Vistula multireservoir system was made by J. Kindler [1977], who took into account the system shown in Figure 2.

Water control objectives, in the (1,2-3,4) system were limited for the purpose of the study to:

a. water supply for five municipal and industrial centers of the region, and
b. maintenance of minimum acceptable flows (MAF) in some of the river reaches.

The mean monthly target water demands and MAF rates and the penalty functions describing economic losses due to not

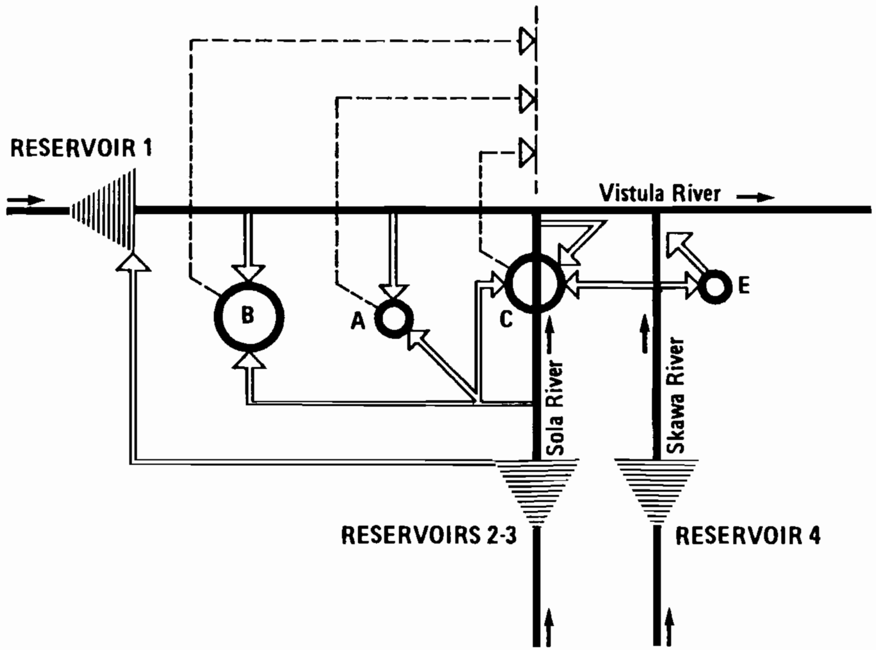

Fig. 2. Simplified layout of the Upper Vistula system (Kindler, 1977).

meeting the target demands and MAF rates have been determined by a separate study.

It was assumed that the system derives its supply from the randomly distributed natural inflows. The inflow into any branch of the system, in any time period, cannot be analyzed in isolation from the other branches or from inflows in other time periods. Since the explicit consideration of the multivariate inflow process poses a number of well-known difficulties in the case of a multireservoir situation, the method approaches the problem in a way consisting of the three following steps:

1. Development of a mathematical model of the multivariate (time and space) river flow process and generation of a synthetic trace of inflows to the system;
2. Development of a mathematical model of a water resources system and simulation of its operation over the long trace of synthetic inflows (simulation coupled with one of the mathematical programming techniques);
3. Statistical analysis of the results of the simulation-optimization computations and identification of the optimal operation rules for the system of storage reservoirs.

Such a procedure, generally known as the Monte Carlo or stochastic implicit approach, was first proposed by Young [1968], for the solution of a single reservoir problem.

3.1 MULTI-SITE FLOW GENERATION

For implementation of the first step of the proposed method, a multi-site flow generation model was developed, based in principle on the paper by Matalas [1967]. The model was based on the following assumptions:

1. The process is a cyclic one and the number of elements correspond to the number of time intervals into which the year is divided.

2. The internal structure of the streamflow time series is described by a lag-one Markov process with a discrete time parameter.
3. The mean flows in each time interval of a year are log-normally distributed.
4. The normalization of marginal distributions, in this case by a logarithmic transformation, leads to normalization of the multivariate distribution [Kaczmarek 1963].

For the case system consisting of three reservoirs, for which mean seasonal (monthly) synthetic streamflows were desired, the subject of modelling was the sequence of three multi-dimensional random variables

$$\{\overline{Q}_2(i),\ \overline{Q}_{2-3}(i),\ \overline{Q}_4(i)\} \tag{1}$$

where the lower index denotes the site number (Figure 1 or 2) and i (i = 1,2,...,n) denotes the season number.

Following normalization of sequence (1), several statistics associated with these sequences were estimated. Using these statistics, the model for generating synthetic streamflow sequences was developed for each month of a year.

The statistical resemblance between the historic and synthetic streamflow sequences was analyzed by the t-test (mean values), z-Fisher test (variances) and log-transform test (correlation coefficients). At the significance level of $\alpha = 0.05$, the differences between the corresponding statistics proved to be insignificant for about 90 per cent of the analyzed parameters.

As a result of the streamflow generation, 100-year-long synthetic sequences of mean monthly inflows to the system were obtained.

3.2 SIMULATION-OPTIMIZATION MODEL OF THE SYSTEM

The next step of the approach presented here was to develop a simulation model of the system, where at each time step the vector of optimal controls is defined by application of one of the mathematical programming techniques. In the case of the multireservoir water resources system, this vector consists of optimal releases from individual reservoirs. The "optimality" of decisions on the reservoir releases depends to a large extent upon the forecast of future inflows to the system. The problem arises of how many future inflows influence the decisions concerning the reservoir releases at a particular moment. In other words, the question is what is the time-horizon of significant future inflows when the reservoir operator must make his decision concerning the release.

Referring to the Upper Vistula situation where reservoir capacities are relatively small, it has been assessed that, for the monthly seasons, the "significant future" is equal to approximately six months [Hydroprojekt 1972]. Therefore the basic assumption which underlies the simulation-optimization analysis is that future inflows to the system--those which influence the decision on reservoir releases--are known. All inflow values are elements of the previously generated synthetic traces and the simulation-optimization process is carried out in a deterministic environment.

At the first step of the simulation procedure the following optimization problem is solved: for the given vector $\overline{V}(1)$ of initial storage volumes in all reservoirs, for the given sequence of vectors $\overline{Q}(i)$ of inflows to the reservoirs in the $i = 1,2,\ldots, 6$ months, and for the given pattern of water demands in the water resources system served by these reservoirs (for $i = 1,2,\ldots,6$), define a sequence of vectors $\overline{W}(i)$ of final storage volumes in all reservoirs that minimizes the total cost of operating the system. The vector $\overline{W}(1)$ was next used as the initial storage volume for the solution of similar optimization problems (with appropriately changed reservoir inflow and demand data) for the next 6 months. This simulation-optimization process was carried out for the whole 100-year period for which synthetic sequences of mean monthly inflows to the system were generated earlier. The cost of operating the system was the function of water transfer pumping costs and penalties associated with not meeting the predetermined water demands as well as the minimum flow requirements in the system.

It should be noted here that the sequence of vectors $\overline{W}(i)$ describing optimal reservoir volumes at all time intervals of the simulated period correspond to the $\overline{D}(i)$ sequence of vectors describing optimal releases from the reservoirs. These releases are optimal from the point of view of the minimized objective function subject to a set of three reservoir balance equations and other constraints.

The optimization problem can be rewritten in a short, mathematical form as given below:

$$\min_{\overline{W}} K(\overline{W}) \tag{2}$$

subject to:

$$\overline{W}(i) = \overline{V}(i) + \overline{Q}(i) - \overline{D}(i); \quad i = 1,2,\ldots,k \tag{3}$$

where:

$\overline{V}(i)$ - given initial vector of states of the reservoirs,

$$\overline{W}(i) = \overline{V}(i + 1) \tag{4}$$

and to a set of respective inquality-type constraints on decision variables. Function $K(\overline{W})$ is the objective function of the model.

The simulation-optimization model employed the out-of-kilter algorithm (e.g., see Fulkerson [1961] and Barr et al. [1974]) which is a special purpose linear programming method designed for the solution of network allocation problems. The suitable implementation of the algorithm allows for dynamical generation of reservoir releases, in accordance with the current and forecasted water demand and inflow situation (the operation rules do not have to be specified a priori).

As a result of the second step of the computations, a set of optimal control vectors $\overline{D}(i)$ and $\overline{W}(i)$ was obtained for each month.

3.3 IDENTIFICATION OF THE OPERATION RULES

The sequences of optimal releases from the reservoirs, determined for each of the months and resulting from implementation of the first two steps of the procedure, were used next for estimation of the parameters of the operational rules. It was decided to describe the relationship between the state vector $\overline{V}(i)$ of the system at the beginning of the i-th time period, the vector $\overline{Q}(i)$ of forecasted inflows in the given season and the vector $\overline{W}(i)$ of final storage volumes at the end of the season by a set of linear equations:

$$\begin{aligned} W_j(i) = {} & b_{o,j}(i) + b_{1,j}(i) \cdot Q_1(i) + b_{2,j}(i) \cdot Q_{2-3}(i) + \\ & + b_{3,j}(i) \cdot Q_4(i) + b_{4,j}(i) \cdot V_1(i) + b_{5,j}(i) \cdot V_{2-3}(i) + \\ & + b_{6,j}(i) \cdot V_4(i) \quad , \end{aligned} \tag{5}$$

where j is the reservoir index.

The set of operation rules for the system was defined by estimating the sequence of parameters $\{b_{o,j}(i), b_{1,j}(i), \ldots, b_{6,j}(i)\}$ for each of the reservoirs. The parameters b have been estimated by the linear step-wise multiple regression using the observations on $\overline{Q}$, $\overline{W}$, and $\overline{V}$ obtained as a result of the simulation-optimization procedure.

The set of (12 x 3) regression equations constitutes the operation rules, the application of which can secure the long-term optimality of reservoir operation.

3.4 REMARKS ON SECTION 3

The fundamental assumption for this kind of operation (or decision) rule is the feedback between the state of the system expressed in terms of actual volume of water stored, forecasted inflows and resulting values of releases from the reservoirs. The relationship between information available, which is necessary for decision making, and the final decision on the desired releases from the reservoirs of the considered time interval (season), can be visualized as the implication formula:

$$\{i\} \cdot \{\bar{V}(i),\ \bar{Q}(i)\} \rightarrow \{\bar{W}(i) \equiv \bar{V}(i+1)\} \rightarrow \{\bar{D}(i)\} \quad . \tag{6}$$

The decisions are taken in a centralized manner (even if some parameters of the operating rule are equal to zero) on the basis of the current state $\bar{V}(i)$ of reservoirs and forecasted inflows $\bar{Q}(i)$ to the system. It is worthwhile stressing the fact that the parameters b of the operating rule do not depend directly on the current state of or inflow to the system. They depend first of all on time but also hidden in this relationship is the dependence on statistical properties of the inflow process and the state of the system's reservoirs.

The centralization of the decision rule and high degree of its aggregation makes the decisions quite general and aggregated. Listed above are features of the method presented as well as the length of the time discretization interval, which is equal to one month. These make this approach more applicable for long-term operation planning or target storage volume determination than for real-time control or on-line control purposes. Therefore, the approach presented provides a rule for storage policy planning.

4. OPTIMIZATION OF MULTIRESERVOIR SYSTEM OPERATION RULES VIA THE SIMULATION METHOD

Another approach to development of the operation rule for the Upper Vistula system was presented recently by Slota et al. [1978].

For modelling and simulation purposes the system shown in Figure 1 was divided into three subsystems [Słota 1978]:

1. A subsystem of the distribution of water resources. The subsystem consists of storage reservoirs (1,2,3,4,5), man-made conduits delivering water to the users and the Sola River channel downstream from reservoir (3), cross-section H, and Skawa River (below reservoir (4) to cross-section N);
2. A subsystem of water use, including the most important water users in the system specified as A,B,C,D,E;
3. A subsystem of water quality which is composed of the Vistula River (the river reach between reservoir (1) and cross-section G), the Przemsza River along its main course, and their tributaries.

The relationships among all subsystems and their respective inputs and outputs are shown in Figure 3.

4.1 SUBSYSTEM OF WATER RESOURCES

Problems of water quantity and distribution predominate over problems of water quality which can be neglected when the model of the first subsystem is derived.

There are two vector inputs to the first subsystem:

- vector $\bar{Q}^{(1)}$ describing natural inflows, and
- vector $\bar{X}$ of control variables such as releases from the reservoir and flows in the conduits.

There are two vector outputs from the subsystem:

- vector $\bar{M}$ of water supply to the subsystem of water use and
- vector $\bar{U}^{(1)}$ of releases from the reservoirs or flows in river channels.

These latter streams are inputs to the water quality subsystem.

Elements of another vector $\bar{V}$, express volumes of water stored in a system's reservoirs.

All vectors mentioned above are functions of time but for brevity's sake this dependence is not indicated in the equations which follow.

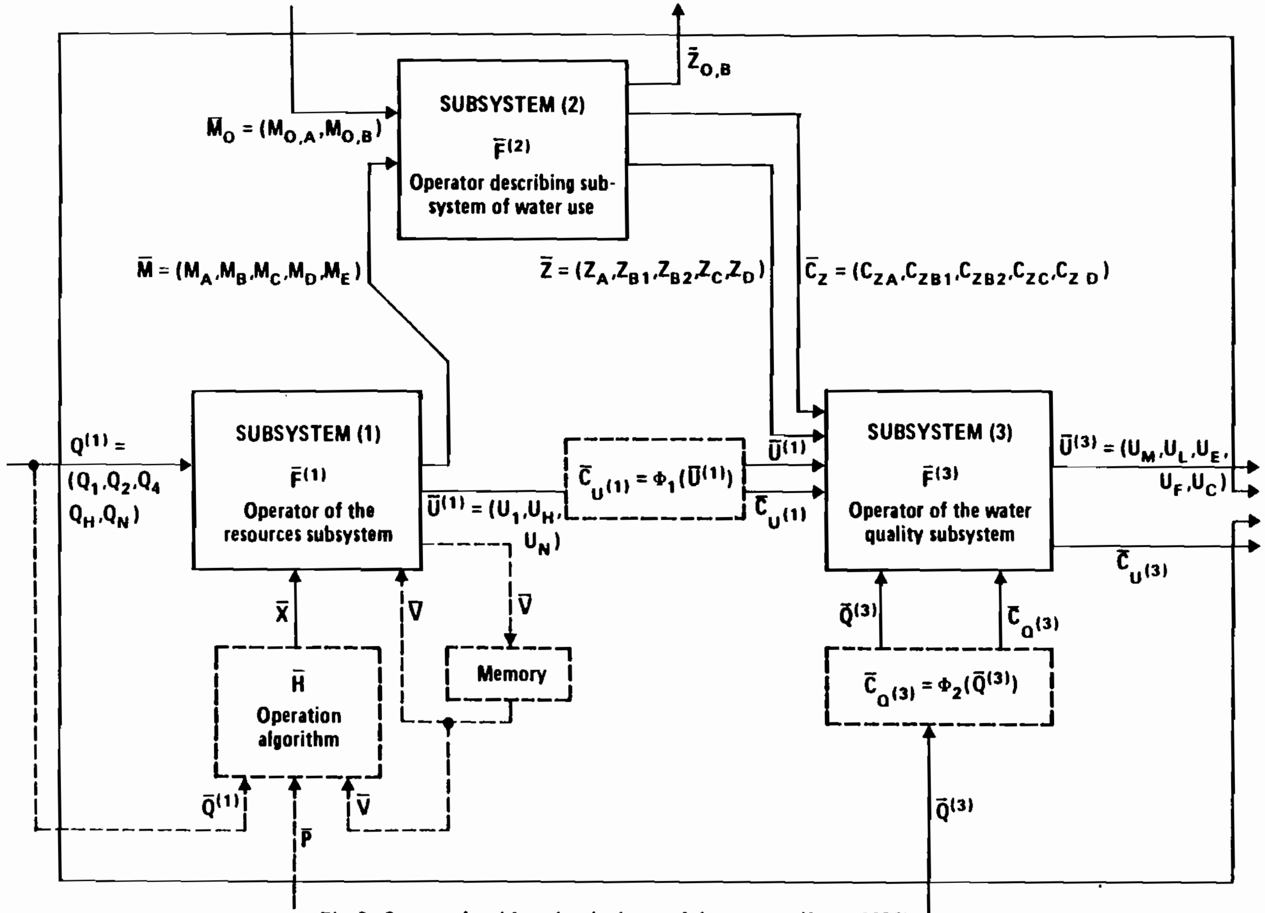

Fig. 3. Structural and functional scheme of the system. (Slota, 1978) (Informational inputs and non-physical elements of the system are marked by the dotted line.)

The flow balance equations which have been formulated for the specified hydrologic cross-sections or for the specified nodes of the system, as well as the reservoir balance equations, are used to describe the processes taking place in the sub-system of water resources.

The relationships among vector variables describing a subsystem's water balance and outputs from the subsystem are expressed by means of the operator $\overline{F}^{(1)}$ given formally as:

$$\{\overline{M},\overline{U}\} = \overline{F}^{(1)}(\overline{Q}^{(1)},\overline{X},\overline{V}) \tag{7}$$

4.2 SUBSYSTEM OF WATER USE

Major difficulties arose when the model of water use subsystem was derived.

A model of this subsystem was used to describe the transformation of its inputs:

- $\overline{M}_o$ = water supply to the users from their own sources or from the system's environment;
- $\overline{M}$ = water supply from the resources subsystem,

into outputs such as:

- $\overline{Z}_o$ = wastewater discharge outside the system;
- $\overline{Z}$ = amount of water discharged to the subsystem of water quality, and
- $\overline{C}_z$ = concentration of selected water quality indices in wastewater discharged to the subsystem of water quality.

Operator $\overline{F}^{(2)}$ of the subsystem describes very complicated processes associated with water treatment, flows in the pipeline network, municipal and industrial water use, wastewater and sludge treatment, precipitation on--and outflow from urbanized areas, etc.

Due to the lack of sufficient information, the model of a water use subsystem is rather general; the amount of sewage and wastewater discharged by water users was evaluated as a function of time-dependent water supply.

The relationship between water supply and wastewater discharge is modelled from the quantitative point of view only. It was assumed that the water quality indices are constant and equal to the mean concentrations which have been evaluated on the basis of measurements performed in 1973 and 1974.

The total amount of wastewater discharged from each particular water user to a particular river basin was derived by the formula:

$$Z_{l,k} = \beta_{l,k} \cdot \alpha_l \cdot (M_{o,l} + M_l) \tag{8}$$

where:

l = index of water user (l = A,B,C,D,E);

k = index of wastewater discharge point;

$\beta_{l,k}$ = coefficient describing partitioning of wastewater amount among separate points of discharge ($\sum_k \beta_{l,k} = 1$);

α_l = reduction coefficient evaluating water losses (including water discharge outside the system);

$M_{o,l}$ = water supply from out-of-the-system sources;

M_l = water supply from the water resources subsystem.

Coefficients α and β were evaluated on the basis of data collected in 1973 and 1974, when all points of water intake and wastewater discharge had been identified.

The identification of the coefficients in the model describing subsystem of water use, stationarity of these coefficients and their dependence on the amount of water delivered to the user--these are the crucial problems encountered in model development. But such problems are caused mostly by the lack of or inaccessibility to suitable and sufficient data.

4.3 SUBSYSTEM OF WATER QUALITY

The model of the third subsystem, where water quality phenomena dominate, attempts to describe qualitative as well as quantitative processes taking place in these river reaches which have been incorporated into the subsystem.

Inputs to this subsystem are given as outputs from the water resources and water use subsystems; they are also given as vectors $\overline{Q}^{(3)}$ and $\overline{C}_{Q^{(3)}}$ characterizing the quality and quantity of uncontrolled inflows to the subsystem, water intakes and wastewater discharges of water users belonging to the system environment (system-environment interactions are not shown in Figure 1 because of their considerable number).

Vectors $\overline{U}^{(3)}$ and $\overline{C}_{U^{(3)}}$, describing streams at the chosen cross-sections from the quantitative and qualitative point of view, are the subsystem outputs. In the face of the lack of possibilities to directly determine vectors $\overline{C}_{U^{(1)}}$ and $\overline{C}_{Q^{(3)}}$, the dependence between flows and concentration of selected water quality indices was expressed in the model by means of operators Φ_1 and Φ_2 (see Figure 3). These operators have been derived on the basis of historical data. Relationships among processes taking place in the subsystem of water quality can be briefly described by the following equations:

$$\overline{U}^{(3)} = \overline{F}_u^{(3)}(\overline{U}^{(1)}, \overline{Z}, \overline{Q}^{(3)}) \tag{9}$$

$$\overline{C}_{U^{(3)}} = \overline{F}_c^{(3)}(\overline{U}^{(1)}, \overline{Z}, \overline{Q}^{(3)}, \overline{C}_{U^{(1)}}, \overline{C}_z, \overline{C}_{Q^{(3)}}) \quad .$$

The vector function $\overline{F}_U^{(3)}$ describes by means of balance equations the quantitative inter-relationships among inflows, flows, amounts of water withdrawn or wastes discharged. Subsequently, function $\overline{F}_c^{(3)}$ is concerned with two fundamental processes taking place in the river flow--dilution and self-purification--which are described by the classical Streeter-Phelps equation [Adamczyk et al. 1978]. It was assumed that the qualitative processes in the Vistula and Przemsza Rivers can be described with sufficient accuracy by a model consisting of a series of several nodes and elementary intervals representing separate reaches of the river.

Water quality was described using seven indices, such as BOD_5, dissolved oxygen, oxygen consumption, phenols, chlorides, sulphates and suspended solids.

4.4 OBJECTIVE OF SYSTEM OPERATION

The objective of optimal control in the system is equivalent to determination of such a sequence of control variables:

$$\{\overline{X}\} = \{\overline{X}(1),\ \overline{X}(2),\ldots,\overline{X}(i),\ldots,\overline{X}(N)\} \quad , \tag{10}$$

where N is a total number of discrete time invervals during the control period, which satisfy all constraints and secure minimization (or maximization) of the system's performance index (objective function). It was assumed that for each discrete time interval the elements of control variables vector $\overline{X}$ are determined on the basis of fixed and a priori rules of water distribution, with parameters of these decision rules evaluated by means of simulation-optimization techniques.

The operation rule can be represented as an operator $\overline{H}$ defined on the vectors $\overline{V}, \overline{Q}^{(1)}$, $\overline{P}$ and $\overline{V}$:

$$\overline{X}(i) = \overline{H}(\overline{V}(i), \overline{Q}^{(1)}(i), \overline{P}(i), \overline{V}) \tag{11}$$

where:

$\overline{V}(i)$ = state variables vector equivalent to volumes of water stored in each of the reservoirs at the beginning of the i-th time interval;

$\overline{Q}^{(1)}(i)$ = forecasts of natural inflow during the i-th time interval;

$\overline{P}(i)$ = vector of water demands in the system in the same time interval;

$\overline{\nu}$ = vector of unknown parameters of operation rules.

Simulation of the control process in the system, when the decision variables are determined using formula (11), allows selection of parameters $\overline{\nu}$ which secure optimal operation of the system.

Therefore, the results of the simulation of the system operation provide a basis for rational selection of parameters of the decision rules.

4.5 SYSTEM SIMULATION AND OPTIMIZATION OF THE OPERATION RULES

Regulation capacity of the water resources subsystem does not allow 100% certainty in meeting all of the total water demands in the system [Slota and Wawro 1979]. It is obvious that periodic water deficits in the system can occur; therefore the purpose of optimization and control is to minimize and properly distribute in time these deficits. Towards this aim, three groups of users were distinguished, according to their relative importance. This classification has been established arbitrarily but several variants were considered, for example, such as:

- Group I - of the highest priority; this group includes minimum acceptable flows and 75% of the total municipal and industrial water demands;
- Group II - the remaining 25% of municipal and industrial water demands;
- Group III - all other water users.

Classification of water users is done according to the sequence of water supply reduction when the amount of water stored in the system's reservoirs decreases and is not enough to satisfy the total demands.

The first two coordinates of vector $\overline{\nu}$ set up the limitations on the summarized volumes of water stored and predicted inflows to the reservoirs during the nearest month. According to the limitations, water supply to the users is restricted. Therefore parameters ν_1 and ν_2 define three states of the system. For state No.1 water demands of all users in the system are satisfied; at the second state (No.2) only users of Groups I and II are taken into account, and at the third (No.3) state of the system, only users of the highest priority, i.e., those which belong to Group I can be supplied.

The other coordinates of vector $\bar{\nu}$ are equivalent to the parameters of functions defining releases from the reservoirs, flows in the conduits and transfers of water among river basins. The following general assumptions have been introduced:

-- the proportions among outflows from the reservoirs supplying the common balance node in the system is determined by the ratio between volumes of water stored in those reservoirs;

-- the amount of water transferred among river basins is the linear function of the flow at the outlet cross-section of the river and the volume of water stored in the reservoir supplying this cross-section (in case of reservoir (5) the amount of water transferred is also a function of the volume of water stored in this reservoir);

-- for all states of the system, the form of operational rules is the same, the only differences are in the values of parameters;

-- water demands are to be successively satisfied according to the predetermined hierarchy of the users and according to the number of existing sources of water (in the first order minimal flows in the rivers are maintained, then water users supplied from one source of water, afterwards users supplied from two sources, etc.).

The number of parameters $\bar{\nu}$ results from the degree of complexity of the assumed form of the operation rules and functions describing the resource allocation process. For the system presented in Figure 1 there are 22 elements of vector $\bar{\nu}$.

The previously formulated objective of the optimal water resources distribution was treated as the polyoptimization problem.

Vector $\overline{M}$ characterizing water users supply and vector $\overline{C}_{u(3)}$ describing the quality of water in the system have been used to evaluate the results of the system operation. Values of the elements of these vectors have been settled by the choice of operation rule parameters which belong to vector $\overline{\nu}$.

The consequences and effects of the operation have been estimated based upon some statistical characteristics of $\overline{M}$ and $\overline{C}_{u(3)}$ vectors obtained from a computer simulation of the system's operation over a 45-year-long sequence of historical data. Results of the operation are expressed in terms of the following performance indices:

(1) performance index evaluating control in the system from the point of view of meeting water demands:

$$\max_{\overline{\nu}} \left[K_1(\overline{\nu}) = \sum_{j=1}^{3} \alpha_j \cdot G_j(\overline{\nu}) \right] , \tag{12}$$

where:

j = index of the group (category) of water users;

$G_j(\overline{\nu})$ = warranted frequency of meeting water demands of the user belonging to the j-th group;

α_j = the weighting coefficient. Values of those coefficients have been assumed fairly arbitrarily: $\alpha_1 = 99999$, $\alpha_2 = 680$, $\alpha_3 = 2$. These values give preference to solutions which secure a 100% guarantee of meeting water demands of the first group of users and simultaneously maximizes the guarantee of satisfying demands of Group II water users.

(2) performance index evaluating system operation from the point of view of water quality:

$$\min_{\bar{v}} \left\{ K_2(\bar{v}) = \left[\sum_{p=1}^{4} \beta_p \left(\frac{1}{w} \sum_{q=1}^{w} J_q(\bar{v}) \right) \right] \frac{1}{\sum_{p=1}^{4} \beta_p} \right\} \tag{13}$$

where:

p = index of the control cross-section ($p = 1$ for cross-section L, $p = 2$ for cross-section M, $p = 3$ for cross-section F and $p = 4$ for cross-section G);

q = water quality index;

w = number of water quality indices considered;

β_p = weighting coefficient for the p-th cross-section (it was assumed $\beta_1 = \beta_2 = 1$; $\beta_3 = \beta_4 = 3$);

$J_q(\bar{v})$ = mean value of the q-th water quality index in the considered period of time.

Values of water quality indices have been determined in accordance with the proposal of Prati et al. [1972] who developed functions for converting incompatible (between each other) concentrations of pollutants to the comparable values of water quality indices.

The number of control variables and the relatively long time of computer simulations have caused several simplifications to be introduced to the optimization procedure. It was decided to perform the so called stage-optimization. At first, with respect to the performance index (12), simulation was focused only on the problems of resources distribution in subsystem I. The relaxation method was used to search for the maximum value of performance index and during the optimization procedure the range of parameters' variability and the lengths of the searching step have been limited, based on observations of the results obtained.

As a result, the optimal solution, as well as the set of feasible solutions which assure that water demands are satisfied with the same tolerance, were determined.

At the next stage of optimization, the simulation of the system operation over the 15-year sequence of daily mean flows was performed with respect to the qualitative and quantitative processes. In this way the value of performance index (13) was evaluated.

The optimal values of parameters of the decision rules have been chosen according to the compromise approach where the sum of proportional deviations of the performance indices (12) and (13) from their optimal values (obtained from the first two stages of the optimization-simulation procedure) was minimized.

4.6 REMARKS ON SECTION 4.

The method described in this section differs considerably from the Monte-Carlo approach presented in the previous section, despite the similar form of the operation rule which is defined on the basis of a state vector V and forecasts of natural inflow $\overline{Q}$. First of all, the parameters $\overline{\nu}$ of the operation rule (11) are defined on the basis of the current state of the reservoirs, while in the Monte-Carlo approach the vector of parameters b is determined depending on the season and does not depend on the current state of the system. The approaches differ also because of the less general, and therefore more detailed character of decisions which directly result from the operation rule (11). Another difference is caused by the fact that the planning rule (5) was derived without taking into account the problems related to water quality. One of the objectives of the method presented in this section was explicitly expressed in terms of water quality control. The lengths of the discretization time interval (one month) and the assumed form of the planning rule (5) and the operation rule (11) mean that there is no possibility of using short-term forecasts and it is impossible to introduce relatively frequent modifications of the storage policy. It makes the planning or operation rules, described previously, relatively inflexible.

5. HIERARCHICAL STRUCTURE FOR MULTIRESERVOIR SYSTEM OPERATION

The approach which is presented in this section is based on the concepts of hierarchical control systems (see Findeisen, Malinowski [1979]). The general idea of the approach is to introduce a two-layer structure for the control of systems operation. The upper layer of the control structure is responsible for the determination of the storage policy of the reservoirs over the long time-horizon, while the lower layer accomplishes operating rules (to be applied for on-line control) using short-term forecasts.

In the following sub-sections the model of the case system is briefly described, then in a more systematic manner the particular elements of the control problem are discussed.

5.1 MODEL OF THE UPPER VISTULA WATER RESOURCES SYSTEM

A simplified version of the case system, which was presented in Figure 1, is shown in Figure 4 (Salewicz [1978]). Three storage reservoirs are distinguished in the model:

(1) Goczalkowice, (2)Tresna, and (3) Czaniec which in this model comprises two reservoirs: Porabka and Czaniec.

The streamflow rates, their quality, and their mutual relationships are described by means of the reservoir balance equations, the flow balance equations formulated for the selected cross-sections, the pollutants balance equations and the model of the self-purification process in the Vistula River reach between the control cross-sections F and G_1.

Vector V is used to describe the state of reservoirs. Natural inflows to the system are represented as Q_1, Q_2, Q_p, Q_L and Q_H. The decision (control) variables are water supply rates for the specified users: $M_A, M_{B1}, M_{B2}, M_D, M_C, M_R$ and also releases from three reservoirs: U_1, U_2, U_3. The wastewater discharges, denoted by Z with the respective subscript, are handled as the external variables (forecasts). The simplifications of the model do not reduce the generality of the control scheme which is described below, and the extended, more detailed model

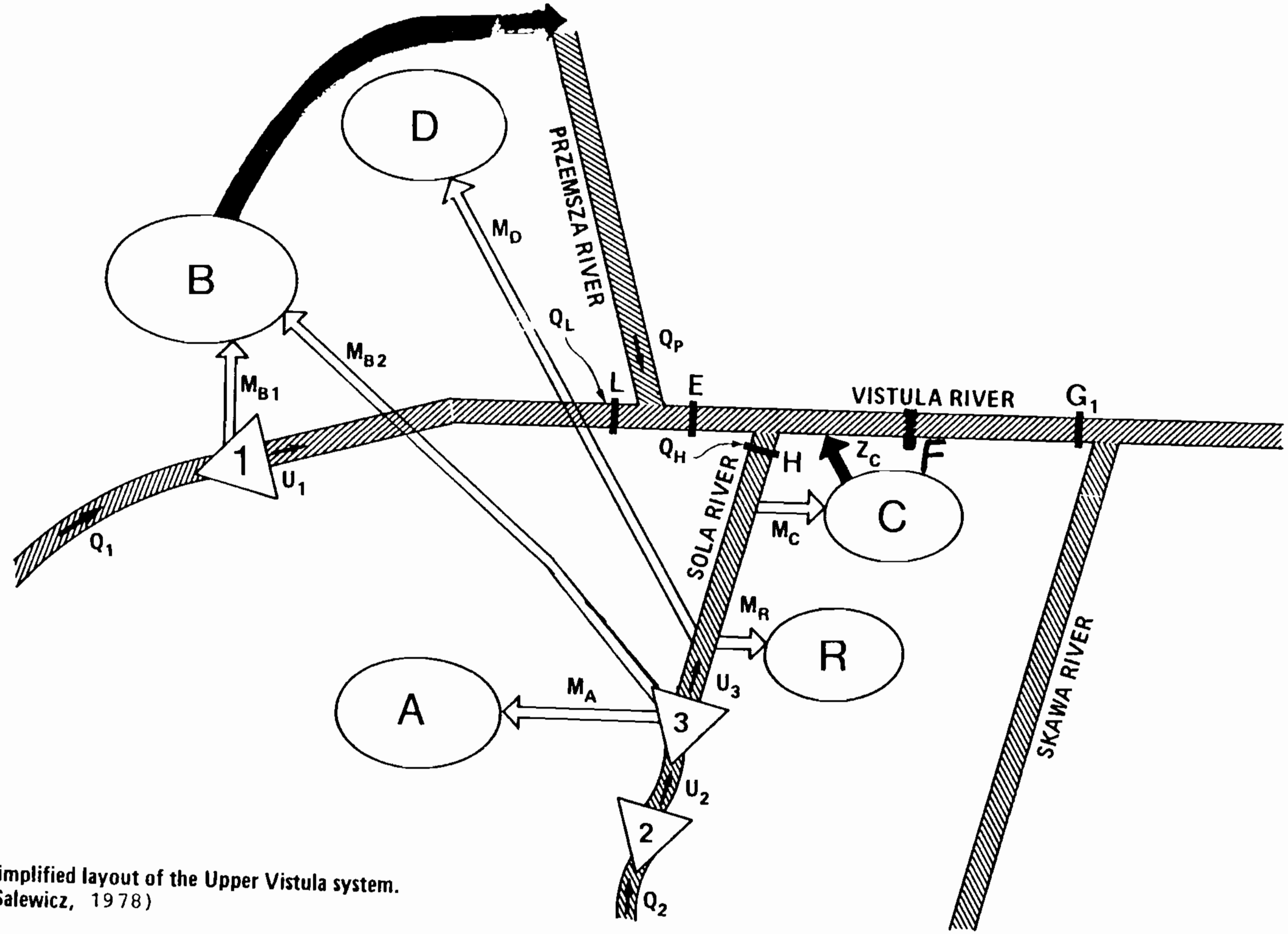

Fig. 4. Simplified layout of the Upper Vistula system. (Salewicz, 1978)

of the investigated water system can also be adopted to this scheme.

5.2 THE BASIC CONCEPTS OF THE PROPOSED CONTROL SCHEME

The control system (Figure 5) has, as was stated previously, a two-layer structure (see Kaczmarek et al. [1978], Malinowski, Terlikowski [1978], Terlikowski [1978]). The upper layer determines the storage policy $\hat{V}(t)$, $t \varepsilon [t_o, t_f]$ ($[t_o, t_f]$ is the optimization horizon), of the system on the basis of the following information:

-- measured, current state $V(t_o) = [V_1(t_o), V_2(t_o), V_3(t_o)]$ of the reservoirs;

-- long-term forecasts of natural inflows to the system (vector Q (t)) and water demands described by means of the time dependent vector function P(t) characterizing water demands of specified users. These vector functions are defined over the planning horizons $[t_o, t_f]$, e.g., 3 or 6 months) and when the discretized version of the model is considered, vectors Q(i) and P(i) denote the mean weekly value of the flows' intensity.

The planned trajectory $\hat{V}$ (t) results from the solution of the following, general form of the dynamic optimization problem:

$$\min_{M,U} \int_{t_o}^{t_f} K(M(t), U(t), P(t), Q(t), Z(t))\,dt + K_V(V) \qquad V\ (14)$$

subject to:

-- so called "local constraints" given as

$$(M(t), U(t))\ \varepsilon\ MU, \qquad (15)$$

-- "global constraints":

$$v(t)\ \varepsilon\ V(t),\ t\ \varepsilon\ [t_o, t_f]\quad , \qquad (16)$$

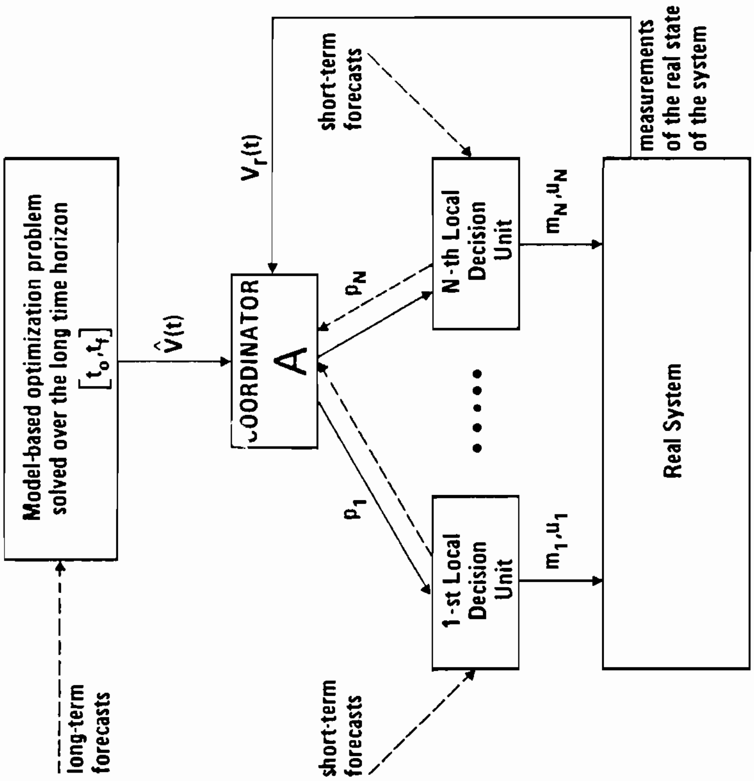

Fig. 5. Scheme of the control structure.

where MU and V(t) are specified sets of constraints (given for example as the balance equations, inequality constraints, etc.).

The objective function K is expressed in terms of the penalties associated with unsatisfied water demands, minimal acceptable flows and desired water quality standards. It can be decomposed with respect to individual water demands and objectives of the system operation:

$$\begin{aligned} K(M,U,P,Q,Z) = {} & K_A(M_A,P_A) + K_B(M_{B1},M_{B2},P_B) + \\ & + K_C(M_C,P_C) + K_D(M_D,P_D) + K_R(M_R,P_R) + \\ & + K_{U_1}(U_1) + K_{U_2}(U_2) + K_{U_3}(U_3) + \\ & + K_q(U_1,U_2,U_3,M_C,M_D,M_R,Z_C,Z_P,Q_H,Q_L,Q_P) \quad . \end{aligned} \tag{17}$$

The latter component, K_q, expresses the "losses" associated with exceeding the desirable concentration of the pollution indices of the control cross-sections E and G_1.

The simplified static formulas, describing water flow balance and self-purification process are included in the expression defining function K_q. The local constraints (15) can be decomposed analogously to the objective function decomposition.

One of the most relevant questions is concerned with defining the function $K_v(V)$ [see (14)] and global constraints (16). These questions can be relatively easily answered as far as intermediate values of V(t) (i.e., for $t < t_f$) are determined; however, the key question concerns the value of the final state $V(t_f)$ which should be given as a fixed target point (stiff constraint) or as the desired one, introduced to the function $K_v(V)$ as the penalty-type function of the deviations of the final state from the target value.

This value is the most important parameter with regard to the dynamics of the system ("the distant future"). If the optimization horizon $[t_o,t_f]$ is relatively short, then the value of $V(t_f)$ could be defined, for example by simulation-optimization techniques; by application of some ideas which lead to the determination of the planning rules described in Sections 3 and 4.

The formal difference between upper layer activity and methods presented in Sections 3 and 4 consists of an explicit definition, as a given time dependent function $\hat{V}(t)$, of some trajectory over the planning (optimization) horizon $[t_o,t_f]$. The practical difference results from the fact that the upper layer of the hierarchical control structure has a more elastic construction which allows change in the priorities of control (optimization), constraints and parameters in a relatively simple manner.

5.3 THE LOWER LAYER OF THE CONTROL STRUCTURE

The objective of the lower layer is to generate direct control decisions M(t), U(t) according to the storage policy determined by the upper layer at the beginning of the time interval (optimization horizon) $[t_o,t_f]$. The information which is necessary for determination of the controls is the following:

-- an actual (measured) state of the reservoirs V(t), and
-- short-term forecasts (e.g. twenty-four hours, one week) of uncontrolled phenomena such as natural inflow, water demands, etc.,

The operational purpose of the lower layer is to make rational current decisions on water resources allocation with regard to the information used (such as was mentioned above) and given long time-horizon storage policy. Thus, the mechanism of modifying the storage policy is not incorporated in the operating rule. The lower layer is constructed only to improve on-line system operation without the necessity of repeating the long-horizon optimization. The structure of the lower layer allows use of the current information, (i.e., short-term forecasts) in a very elastic way. First of all, the existing structure of the controlled system (with respect to available information and/or decision competences) is considered.

The following decomposition of the considered model of the system was assumed:

-- Subsystem I - which consists of water user B and the upstream Vistula River from the reservoir (1);

-- Subsystem II - two Sola River reaches: between reservoir (2) and (3), and upstream from the reservoir (2);

-- Subsystem III - water user A;

-- Subsystem IV - water users C,D,R, reach of the Sola River downstream from the reservoir (3) and Vistula River downstream from the reservoir (1).

This decomposition implies the existence of four <u>local decision units</u> (LDU), I to IV, associated with the respective subsystems. Each of the local decision units has at its disposal current (the most precise) information concerning the respective subsystem and the set of local decisions. The information pattern and authority range of the particular LDU is shown in Table 1.

Table 1. Information pattern and authority range of local decision units.

LDU	Local information	Local decisions
I	Q_1, P_B	M_{B_1}, M_{B_2}
II	Q_2	U_2
III	P_A	M_A
IV	$P_C, P_D, P_R, Q_E, Q_H, Q_P, Z_C, Z_P$	M_C, M_D, M_R, U_1, U_3

It is worthwhile noticing that the last subsystem (and the corresponding LDU IV) is the most extended, since it includes the river reaches where water quality requirements should be met. Water quality is influenced directly by all control variables and inflows mentioned in Table 1, thus there is a common local objective concerned with all these variables together. Each of the LDU's takes into account its local information and the so-called coordination variables, which are the decisions settled by the coordinator--a central decision unit which instantaneously influences all LDU's.

5.4 COORDINATION OF LOCAL DECISION UNITS

Assignment of the coordinating variables (denoted by p) relates the planning policy derived by solving the long-horizon optimization problem with the on-line control performed by local decision units (in the approach presented in Section 4 this relationship was accomplished directly). The coordinator influences all LDU's in such a way that the whole, controlled system follows the long-horizon storage policy. The LDU operation rule means making an independent, rational allocation of water resources inside the subsystem with regard to local objectives, local short-term forecasts and coordinator decisions p. The local objectives should be properly modified by the coordinator decisions which should also be chosen properly (i.e., an adequate coordinating rule has to be determined as mentioned at the end of this sub-section).

This is the general concept of the proposed on-line control scheme (Malinowski, Terlikowski [1978]). In the current investigations (Kaczmarek et al. [1978], Terlikowski [1978]) based on the optimizing scheme of the control structure, the price mechanism was used as the method of coordination. The coordination variables have been expressed in this scheme as the vector:

$$p = (p_1, p_2, p_3)$$

where p_1, p_2, p_3 are scalar variables, so called <u>prices</u>, corresponding to reservoirs (1), (2), and (3) respectively.

Modified, local objectives (denoted by $L_I, L_{II}, L_{III}, L_{IV}$) for each of the subsystems result from the decomposition of Lagrangian

$$L(p,M,U,P,Q,Z) = K(M,U,P,Q,Z) + < p, \dot{V} > \tag{18}$$

where $\dot{V}$ is the right hand side of the reservoir state equation (for example, the state equation of the reservoir (3) is the following):

$$\dot{V}_3(t) = (U_2(t) - M_A(t) - M_{B2}(t) - U_3(t))$$

It is clear, that

$$L = L_I + L_{II} + L_{III} + L_{IV} \quad ,$$

where $L_I, \ldots, L_{IV}$ depends only on decisions and forecasts related to the respective subsystem; it depends also on p. For example, the modified performance index of subsystem I has the following form:

$$L_I(p_1, M_{B1}, M_{B2}, Q_1, P_B) = K_B(M_{B1}, M_{B2}, P_B) +$$

$$+ p_1 \cdot (M_{B1} - Q_1) + p_3 \cdot M_{B2} \quad .$$

Thus, the LDU's operating rule includes optimization of the local objectives:

$$\min_{M_{(i)}, U_{(i)}} L_{(i)} \quad ; \quad (i) = I, \ldots, IV \tag{19}$$

subject to given prices (coordination variables) and actual, local forecasts.

Prices p should be chosen in such a way, that controls $M(t)$, $U(t)$ defined by the respective LDU according to (19) yield some trajectory $V_r(t)$ of the reservoirs' state, which approximately (according to specific requirements and conditions) follow the storage policy $\hat{V}(t)$ defined at the upper layer.

Based on the measured, actual state of the reservoirs $V_r(t_j)$ $(t_j \in [t_o, t_f])$ the prices are modified subject so some formula A (see Malinowski, Terlikowski [1978])

$$V_r(t_j) \xrightarrow{A} \tilde{p}_j \tag{19a}$$

The coordinating rule (19a) and the LDU operating rule (19) constitute the whole operating rule as accomplished by the lower layer of the considered control scheme.

5.5 REMARKS ON SECTION 5.

At the end of this section some additional properties of the described method are discussed.

Modifications of the local objectives introduced by the specific [see (19)] choice of the prices enable a balancing of, in a rational mann r, current water demands and other requirements within the possibilities offered by the local short-term policy of the system.

It is worth observing that the LDU rule (19) has the following optimality property:

-- if short-term forecasts prove to be fully consistent with reality, then the controls $M(t)$,$U(t)$ assigned by (19) are strictly optimal for the performance index (17), subject to constraint $V(t) = V_r'(t)$, where $V_r'(t)$ is a state trajectory occurring in the real, controlled system.

Hence:

-- local decision units accomplish a rational (optimal) current water distribution in a system;

-- the coordinator sees that this distribution is realized according to the storage policy;

-- the upper layer (planning layer) aims for a proper choice of this policy.

The scheme presented is quite general and elastic. This very natural structure can be adjusted to specific, real conditions. It is possible to introduce some additional elements (which may exist in reality) into the operating rule and use various methods of long-horizon policy planning.

At the present stage of research (when the numerical experiments are extensively performed), the proposed control scheme has not yet been entirely adapted to the system, especially from the practical point of view. However, it seems that its appealing features (as far as control during drought and normal flow conditions are concerned) provide considerable incentive to carry on this research.

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STOCHASTIC MODELING OF MULTI-PURPOSE RESERVOIRS--ONLINE OPERATION

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1. Introduction

The Sonderforschungsbereich 81 at the Technische Universität München is an interdisciplinary research program about problems of the qualitative and quantitative run-off-processes in the alpine and prealpine region. This research program is sponsored by the Deutsche Forschungsgemeinschaft since 1974. At present ten projects are established. The aim of this Sonderforschungsbereich is to evaluate the basis for the optimal use of the natural debit in an over regional water-management system.

Within this research program, the project C2 "Reservoirs in River Systems" is concerned with models for the description of the run-off regulation through reservoirs, the determination of the effects of reservoir operation and their optimization. The main points of this project are the testing of available reservoir models on existing plants, the development of models for the description of the regulation of reservoirs which can be used as a basis for online operation, and the development of those parts of an integrated model for multi-purpose reservoirs which are necessary for optimal planning. Although, this project is part of a regional research program, the validity of its results is not restricted in any way.

2. Long-term reservoir models

Stochastic models for reservoirs consist of a stochastic model for the inflow and a deterministic model for the regulation. Because of the various methods for the solution of reservoir problems and the different purposes of reservoirs, several models for the description of reservoir operation have been proposed.

2.1 General survey

Available models for reservoir operation can be classified under the aspects of the system equation and of the type of operating policy.

Let ΔT be the time-unit in the model, e.g. month or day, then the physical interrelation of the reservoir process are given by

$$S_{i,j+1} = \max\{\min\{S_{i,j} + Z_{(i,j)} - A_{(i,j)}\} K \;, 0\} \tag{1}$$

with

$S_{i,j}$ $[m^3]$ content at the beginning of the j-th time unit ΔT in the year i

$Z_{(i,j)}$ $[m^3]$ inflow during the j-th time unit ΔT in the year i

$A_{(i,j)}$ $[m^3]$ release during the j-th time unit ΔT in the year i

if inflow and release are simultaneous processes. In the cases that inflow occurs before release or vice versa, the system equations are

$$S_{i,j+1} = \max\{\min\{S_{i,j} + Z_{(i,j)}\;, K\} - A_{(i,j)}\;, 0\} \tag{2}$$

inflow before release

$$S_{i,j+1} = \min\{\max\{S_{i,j} - A_{(i,j)}\;, 0\} + Z_{(i,j)}\;, K\} \tag{3}$$

release before inflow

It is evident, that the three types of the system equation describe different physical conditions. An interpretation of these equations as formal descriptions of different basic operating schemes for reservoirs is possible only in those cases where the inflows can be regulated. But the assumption about reservoir operation involved in equation (2) and (3) are very restrictive.

For the formal description of reservoir operation, different types of operating policies are used. One possible classification is the following :

- stationary operating rules

  The release $A_{(i,j)}$ is defined as a function of parameters, e.g. present content $S_{i,j}$ or/and preceeding inflow $Z_{(i,j-1)}$, which are known at the beginning of the time unit under consideration. [4,7]

- adaptive operating rules

  In this case, the release $A_{(i,j)}$ is determined during the respective time unit. Examples for this type of operating rules are those, where the release only depends on the actual inflow or on the actual contents during the time-unit. [5, 7, 8, 17]

- mixed operating rules

  The principle of this type of operating rules is that the release $A_{(i,j)}$ is determined in two steps. Based on the available information, the first decision about the release is made at the beginning of the time unit. During the time unit, the release is adapted according to the actual inflow.

The explicite formulation of the operating rules can depend on the special aspects of the reservoir problem under consideration. In particular, this holds for elements of a multiple reservoir system.

2.2 Results of the model analysis

Based on former investigations [2, 6], the applicability of the various operating models has been investigated enquiring whether the different assumptions agree with the situation of existing reservoirs and how the assumptions influence the statements about the reservoir operation. The results obtained from the analysis for about fifteen reservoirs which are different in the purposes, in the characteristics of the catchments and the integration in systems show that the operation of these reservoirs can be described by

$$A_{(i,j)} = \alpha_j + \beta_j S_{i,j} + \gamma_j Z_{(i,j)} \tag{4}$$

Comparisons of the real world conditions of the reservoirs and the results obtained with different operating models show, that neglection of the content or the actual inflow can cause a high methodical risk for realistic statements about the reservoir operation [8].

Theoretical as well as practical investigations show that some adaptive operating rules depending on the content are special cases of mixed operating risks with the parameters initial content $S_{i,j}$ and actual inflow $Z_{(i,j)}$. Mixed operating rules can be applied in stochastic queuing models, stochastic simulation models and optimization models [7, 9, 11, 12].

As special property of mixed operating rules with a linear inflow component is that the long-term operating can be transformed in an equivalent "online" rule. It is

$$a_{i,j}(\tau) = (\alpha_j + \beta_j S_{i,j}) / \Delta T + \gamma_j z_{i,j}(\tau) \qquad (5)$$

with

$a_{i,j}(\tau)$ [m^3/s] release at the time τ

$z_{i,j}(\tau)$ [m^3/s] inflow at the time τ.

This enables the consideration of short-term restrictions for the release during the time unit T in the "online" operation and in optimization models.

3. Consideration of short-term aspects in long-term reservoir models

The difficulties in the consideration of short-term aspects in long-term reservoir models are caused by the time discretization. In general, the time units are great in comparison with the duration of short-term events. Therefore, the theoretical possibilities of some operating schemes [10, 17] cannot be used directly in practice.

3.1 Integration by special functions

In the cases of minimum-flow restrictions, special functions are used in order to describe the interrelation between the inflow and the release during the time unit and the required or usable amount of water due to this restriction [10, 14]. Whether such functions are necessary depends on the length of the time unit [14].

3.2 Use of small time units

Some models for the generation of the inflows in small time units are available [18]. Their application in reservoir models lead to an essential increasing of the computer requirements in time and storage. Further, the applicability of these models seems to be restricted to special hydrological conditions in the catchments [9].

3.3 Integrated models

Integrated Models for multi-purpose reservoirs have been proposed in various versions. One of them is the use of a daily operating scheme in a monthly model [12]. An other one is the differentiation between month and without floods. In months with flood, the inflow and the operation of the reservoir are represented by special models [1]. While the previous mentioned models work with a uniform time-discretization, an other approach is based on a variable time discretization. The definition of the single time intervals depends on hydrological and operational aspects. Models of this type has been applied successfully.

4. Combined long-term - and flood - model

The combined model for the consideration of floods in a monthly stochastic simulation model for the operation of reservoirs consists of two basic models. These are the monthly model for the long-termed operation of the reservoir, and the hourly model for flood hydrographs and the operation during the flood. The integration of the short-term model in the long-term one is done by a special superposition procedure.

4.1 Monthly model

Various models are available for the generation of synthetic time series of monthly inflows. We applied the Fiering-model; without any difficulties any other model for the generation of monthly inflows can be used.

4.1.2 Operation

Assuming inflow and release as simultaneous processes, the monthly change of the reservoir storage is given by equation (1). During every month, the release $A_{(i,j)}$ is determined by a mixed linear operating rule, equation (4).

4.2 Flood model

4.2.1 Synthetic flood hydrographs

A major part of the investigation in the field of flood-protection with statistical methods is concerned with the problem of suitable general probability function for the flood parameters. As the parameters are physically inter-related a multi-dimensional probability distribution is required to describe flood events. In general, the estimation of such a probability function is nearly impossible.

By our method, floods are defined as inflows which exceed a given truncation level TL. The essential parameters of flood events are the peak flow of the flood P (m^3/s) , the volume of the flood V (m^3) , the duration of the flood O(h) and the relative position of the peak in the flood PP (-).

If the parameter PP is stochastic independent from all the others, the generation of synthetic flood hydrographs is based on the separate generation of this parameter and of the leading parameter of the other ones. The latter ones are deduced by a sequential regression, e.g.

$$V = f_1 (P)$$
$$D = f_2 (V)$$

where the peak flow P is the leading parameter.

The flood hydrographs of the synthetic floods are approximated by a parabola for the increasing part of the flood and a parabola for the decreasing part of the flood [15].

4.2.2 Operation

During the occurence of floods a reservoir can be managed controlled or uncontrolled. The most common release - strategy is the definition of an outflow not to be exceeded. Additionally in our models the controlled outflow is allowed to be implicitly dependent on the inflow - thus, the release - strategy can be different for the rising and the falling limb - the content of the reservoir or both. With non controllable draw - off structures it is essential to bear in mind that the total release of such structures with respect to a certain water level equals the minimum outflow as soon as this water level is exceeded.

After the development of suitable procedures the method of simulation can be used to calculate the probability of failure for specific designs of the reservoir, to investigate the consequences of managing strategies and necessary restrictions. For example, our model computes the interaction between the maximum outflow, the corresponding content and the probability of failure for each given release-strategy. Whenever the reservoir overflows due to one of the, say, one thousand simulated floods the maximum outflow is increased by a defined discharge ΔQ. Alternatively, if the absolute maximum flow is reached, the storage capacity can be stepped up by an equivalent ΔJ. Obviously, adjusting the capacity first and combinations of both methods are equally possible [16].

4.3 Combination of the models

Basic assumptions for the combination of the models for the monthly operation and the operation during floods are that can occur either one flood or no flood during a year and that the sequence of years with or without a flood is an independent identically distributed random process.

4.[illegible].1 Simulation of the flood occurence

Denoting the probability of the occurence of a flood during a year by ξ, the occurence of years with and without a flood can be simulated by the following procedure

$$u_i < \xi \qquad \text{there is a flood in the year } i \tag{6}$$

$$u_i > \xi \qquad \text{there is no flood in the year } i$$

where $0 < u_i < 1$ is a uniformly distributed random variable.

Let be η_j, $j = 1,2,\dots,12$, the probability that there is a flood in the month j, the simulation of the month j in the year i runs as follows

$$\sum_{\lambda=0}^{j-1} \eta_\lambda < v_i < \sum_{\lambda=1}^{j} \eta_\lambda \qquad \begin{array}{l}\text{the flood in the year } i\\ \text{occurs in the month } j\end{array}$$

where v_i has the same properties as u_i in equation (6).

The begin $\tau_{i,j}$ of the flood in the month j in the year i is assumed to be random and it is generated by

$$\tau_{i,j} = w_{i,j} \; (T_j - D_{i,j})$$

where $0 < w_{i,j} < 1$ is a uniformly distributed random variable and T_j and $D_{i,j}$ are the length of the month and the duration of the flood, respectively.

4.3.2 Operation during flood months

The operation during flood months consists of three periods. These are the time before, during and after the flood. Until the begin of the flood, the reservoir operation follows the long-term rule. The release during this time can be determined by the integration of equation (5). During the flood, the regulation is defined by a special operating scheme. After the flood, the long-term is valid again. But the release in this period depends also on that during the flood. If possible, the sum of releases in the three periods should be equal to the release defined by the monthly operating rule (4). If restrictions would be violated, the return to the long-term operation occurs during the next time unit. Details of the procedure are described in [13].

4.3.3 Integration of forecasting

The essential property of the outlined combined model is, that its structure is a very close approximation to the "online" operation of reservoirs. Its concept consider that floods are random events. In the case of "online" use of the model, the substitution of the synthetic flood-model by a model for the forecasting of floods is possible without any difficulties. This holds although for the operating scheme, if adapted regulation schemes to successive improved forecastings of the flood are used.

5. Extension to multi-purpose reservoir systems

5.1 General remarks

Interactions of reservoirs in systems can be caused by operational and/or hydrological conditions. The results of our investigations indicate, that the extension of the linear mixed operation rules for such reservoirs is possible. Results obtained by the application of multi-site data generation models show that streamflow series as well as the occurence of floods in river systems can be described by a stochastic model with a sufficient accuracy, too.

5.2 Application

The combined model has been applied to the reservoir Forggensee. This is a multi-purpose reservoir -- hydro-energy production, flood control, low flow augmentation and recreation -- with a following chain of small dams for hydro-energy production. The outlined model together with an operation model for the hydroelectric system can be used as basis for the online operation. Models for the forecasting of floods, for the optimization of the actual flood regulation can be integrated in this combined operation model.

Current investigations are concerned with the application of the stochastic combined model to a system of multi-purpose reservoirs.

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OVERVIEW OF RESEARCH ON OPERATION OF MULTIPLE RESERVOIR SYSTEMS
(Colorado State University activities)

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Initial Remarks. This overview is related only to the operation of reservoirs in general, and of multiple reservoir systems in particular. Planning and design aspects are not covered. The overview presents the philosophies and concepts of approaches, and the research lines followed. No research results are included, pending their publication under the acknowledgment of the proper financial sponsorships. The overview is separated into two parts: (1) Approaches to reservoir operation; and (2) Research activities.

PART I

APPROACHES TO RESERVOIR OPERATION (Condensed by V. Yevjevich from the write-up by V. Yevjevich, W. A. Hall, and J. D. Salas)

1.1 Positions and Approaches to Research

The problems of the best operation of storage reservoirs have occupied professionals from the time the first modern reservoirs were built in the second half of the last century. With time, the operational rules in the form of operational curves, tables or equations (recently) have been developed. Basically, experience with operation of reservoirs, on a trial-and-error basis, has led to various types of operational rules based on the original reservoir objectives and various constraints.

The characteristics of derived operational rules are: (1) A high level of flexibility in decision making; (2) Ability to integrate in the decision making and rules those factors which are difficult to quantify in monetary and other economical terms; (3) Any new experience (large droughts, unusually wet years, exceptional floods, water quality requirements, changes in the reservoir objectives, more data and knowledge on inflows, reservoir seepage and evaporation, water demand flexibility or rigidity, etc.), has usually led to a corresponding modification of the operational rules; (4) The experience with the design of operational rules on the existing reservoirs has been transferred to the new reservoirs by the principles of analogy and adaptations to the new conditions.

The fact that the operational rules approach to operations of most of the reservoirs in the United States and abroad has been continued regardless of a flood of proposals, based on operational research and various numerical computer-oriented optimization techniques, is a good indication that operational rules still do a better job, regardless of some recent slow progress of the limited use of optimization techniques.

The reluctance to accept new mathematical optimization models has been attributed by their proponents to: (1) Human inertia to new ideas, and (2) Inadequate training of the professionals in modern mathematical methods. While there is some evidence that these factors exist, before one accepts them as the cause of the reluctance, the very real possibility that the mathematical optimization does not accurately portray the real problem situation should be evaluated. Experience in other areas suggests that when new ideas, claimed to be an order of magnitude better, are not accepted, there is usually a good cause.

In this case, a hypothesis is advanced here that the proposed mathematical models are not adequately representative of the fundamental problems of reservoir operations, while the operational rules are considerably more satisfactory. Mathematical models are inflexible. A given set of data (with considerable uncertainty involved) must necessarily produce a fixed answer or a decision policy from a given model. Unless that model is developed in such a manner that operator's judgment, based on experience, can be brought to bear at the appropriate step in the analysis, the results are inflexible. Unless that model is developed with the appropriate objective functions (not one but several functions), it cannot produce the optimal policy for the real problem except by mere chance. Unless the non-quantitative aspects can be incorporated by judgment, the results may be seriously in error. Unless the derivatives or rates of changes of various objective functions and constraints accurately represent the rates of change of the physical and social phenomena, they are intended to portray, the results can be seriously in error, regardless of what is the root-mean-square error between the substitute function and the true function. All of the factors contribute significantly to a conclusion that, in this instance, the reluctance to accept mathematical optimization, as a substitute for judgment based on long experience, may be well founded.

In reviewing most of the previous optimization analyses, including those by the writers of this text, it appears that there has been too much concern with the *form* of the model (i.e., decision variables, an invariant objective function and invariant constraining functions) than with the *substance* of the reservoir operational problems. A single objective function is chosen, often an order of magnitude index of economic returns. The function is (and must be) then assumed to be invariant with time and/or various combinations of exigencies that may result. By an invariant function, it is meant that the mathematical rules for calculating the level of objective achievement do not change, regardless of any eventualities. None of these conditions conform to the reality of most operational systems.

Perhaps most important, the mathematical models are *de novo* rather than evolutionary adaptations of existing procedures. Where they are compatible with existing procedures, they do not produce better results (e.g., optimization vs. mass balance analyses) even though it is more sophisticated and more difficult to interpret any proposed or imposed deviations.

For these reasons, a second hypothesis is advanced here, namely that there is a greater probability of improving reservoir operating rules by step-by-step evolution from the existing practices than by *de novo* mathematical modeling in a different context of the problem. Before the existing operating rules approach is discarded, there should be research and field tests on pilot projects to determine full power and utility of these methods so that they reflect the substance of the operational problems. Without arguing that these rules are perfect, it can be asserted that until these capabilities to reflect real situations have been identified, analyzed and incorporated accurately and effectively into the quantitative analysis, substitution of *de novo* mathematical methods has a very high probability of failing to represent an accurate portrayal of the problems and constraints of the real problem.

In particular, the writers' position is that considerably more attention should be given to the design of appropriate objective functions. Despite the fact that the obvious primary objective of virtually each water project is to improve the *assurance* of the availability (or absence) of water in particular times and places, this fundamental objective is nearly always replaced by a crude economic estimation in the form of the objective function. Assurance is frequently imposed by an arbitrary guess for a penalty function. In effect, this guessed answer then uses a sophisticated system of analysis in a crude attempt to see what the guess turns out to be. In other cases, it is used as an arbitrary constraint (or constraints) which require water to be available (or empty space available) some percentage of the time. While better than the guesses on penalty functions, percentage of time below rated output is a very imprecise index of why one wishes to avoid such deficits in the first place.

The computer-oriented algorithms of optimization start with assumed objective functions for the operation of reservoirs. How these objective functions are obtained is most often irrelevant to the analyst. There also seems that the formalism is put ahead of the substance. The methods and well-defined procedures are needed to determine the objective functions, in relation to various policy criteria and an abundance of the restrictive constraints of general or particular nature. If the various cases of practical optimization attempts have paid much more attention on how to design the most appropriate objective functions, in the light of various changes and developments related to the operation of reservoirs, more meaningful results would be obtained from these attempts. The results of optimizations or operational rules cannot be any better than the reliability of the objective functions would permit. It is likely that the most important losses in the operation of reservoirs, in comparison with their possible optimal benefits, are coming from the inadequate definitions and specifications of the objective functions, rather than from the use of operational rules instead of various optimization algorithms.

The objective functions are time dependent, meaning on the various evolutions with time in water inputs, losses, demands, constraints, and additions of new reservoirs. Therefore, a well-developed methodology, with clear criteria and procedures on how to determine the objective functions, at each important turn of changes in reservoir objectives, in perceiving these objectives or in systems' information, would be used many times over for a reservoir or a system of reservoirs. With some ongoing resistances in USA to the construction of new reservoirs, two new aspects of reservoirs become important: (1) A more advanced and reliable analysis of benefit and cost, as well as various impacts and safety of new reservoirs, will be needed in order for them to be accepted and approved by all the decision making levels, and (2) The objectives of the use of the presently existing reservoirs will be under stress for a change (a reallocation of reservoir objectives), as the untapped water resources become either exhausted or uneconomical for further development and control.

Another basic line in developing new optimization techniques is related to the joint operation of several or many reservoirs, as the multiple reservoir planning and operation. A basic concept followed in the research activities runs something like this: (1) Divide the useful storage capacity of each reservoir into discrete layers as the multi-state division of this capacity, and (2) Operate a system of reservoirs as the multi-reservoir, multi-state and multi-stage optimization problems, with each reservoir having its own objective function, and each layer (state) considered as the discrete state (center of the layer, with all other water storage volumes within a layer considered equal to its center volume value). This approach provides so many combinations or permutations of reservoirs and their states, not to include also the multiple stages (time intervals), that it automatically puts a restriction on the number of states (layers) and/or stages (intervals). It then introduces initially a limitation in accuracy of results by using too coarse a division of storage capacity in only 4-6 states, including the full and the empty reservoir states, or too coarse a division in time.

Because many reservoirs have true objective functions that are either indentical in their compositions, or they follow parallelly similar operational patterns, the concept of a small number of equivalent reservoirs seems a plausible approach in order to reduce significantly the dimensionality in comparison with the reservoir-number and storage-state multi-dimensional approach. Therefore, grouping reservoirs by some criteria into a small number (say one, two, three and a maximum of four) of equivalent reservoirs may significantly simplify the multiple reservoir operations without significantly losing the accuracy of the final results. An equivalent reservoir is defined as a reservoir of such a capacity that it will produce the same results in water released, hydroelectric power produced, flood control accomplished, etc., as would be obtained from a number of reservoirs included into this equivalent reservoir concept.

Criteria, analysis and integration of constraints, storage capacities, losses, inputs, outputs, production, etc., are needed as the new methodology for computing these equivalent reservoirs. Once the decisions are made on how to operate these equivalent reservoirs, either by operation rules or otherwise, the distribution rules would be established on how to allocate the equivalent reservoir decisions to the decisions of real individual reservoirs. This concept of equivalent reservoirs is a promising alternative for the multi-reservoir, multi-state and multi-stage approach to the operation of multiple reservoir systems.

1.2 Operational Rules

The decisions to be made in real time day-to-day operations of a reservoir or a system of reservoirs can basically be characterized by the quantity of water to be released from each storage unit in the immediately following time interval. These releases consist of two parts. This first is the *required release*, predetermined in advance by contract or other express or implied obligation to provide a minimum or assure level of service or avoidance of losses. The second is the *optimal release* corresponding to supplement water, energy or flood control reservation which will enhance the various purposes of the reservoir system. Note, however, that *required releases* are not actually mandatory, but rather are guides which the operator must follow or have very good reasons for deviating.

The net effect of any release decision is: (1) An impact on the economic and other socio-environmental objectives, and (2) A modification of all the risks that certain undesirable states of affairs may occur at some future time. In general, these two classes of effects are in conflict to a greater or a lesser degree in each decision to be made.

Past practice has been to establish *rule curves* or, in general, the *operational rules*, amounting to predetermined judgment concerning the allowable trade-off between the immediate gain and all the future risks. These *rule curves* or *operational rules* are largely judgments or integration of operations based on experience, with the system and its demand backed by such quantitative models as were available and/or relevant.

In recent years, a considerable number of new quantitative models has been advanced, which, according to their authors, can be utilized for real time operational decisions without the necessity of rule curves or some other forms of operational rules.

To the extent that more deterministic information is usually available at the time of the decision than is available in advance, when the operational rules are determined, there is a justification for the use of accurate mathematical models which can make use of the additional foresight it affords. However, to the extent that the models used do not reflect all of the objectives properly, particularly the risks avoidance objectives, the mathematical exercise can be quite counterproductive. Most of the proposed models use economic returns as the objective and attempt to compensate for risks by the imposition of arbitrarily defined penalties assessed whenever water or energy cannot be provided due to prior errors (or when excess water results from infringement on flood storage reserves). Unfortunately, because of the basic nature of optimization methods, this is exactly equivalent to an arbitrary adjustment of the operational rule recommendation. The ensuing analysis then is not an optimization but rather is little more than a calculation in order to determine what the arbitrary guess would have produced. It would appear much more rational in such cases to exercise the judgment directly on the allowable deviation rather than on a penalty accurate in little more than an ordinal sense.

The problem should be treated as a multi-objective decision process with at least two classes of objectives, economic and service assurance. The former can be aggregated to some extent and still be consistent with the standard planning practice. The latter is more difficult and may initially require two or more characterizations to represent large shortfalls for short periods of time as contrasted with small shortfalls for extended periods of time.

Basic to the analysis will be the development of suitable methods for the evaluation of the changes in the risk (or assurance) probabilities which would result from deviations from the planning operational rule as a function of all the information known or obtainable at the time of the decision. With these changes, and the corresponding optimal marginal return from making the changes, the set of trade-off functions between marginal economic returns and marginal changes in risks can be evaluated. These trade-off ratios can then be used in an appropriate procedure, such as the surrogate worth trade-off method (SWT). This analysis, however, is concerned primarily with the accurate objective changes in the risk objectives and economic objectives due to any adjustment of the operation rule decision rather than with the question of whether that risk change is acceptable or not to those concerned.

1.3 Methods Used for Solving Storage Problems

At present, practical bridges do not exist between the general theory of water storage (or storage in general) and the practice of planning, design and operation of reservoirs. There may not be an example in the world for which the size of reservoir storage capacity has been determined basically by the theory of water storage. Three general groups of methods exist for studying the storage problems: the present-day engineering methods, the experimental or generation of new sample methods and the analytical methods.

The *engineering methods* are based on the use of historical data for inputs by determining the storage capacity of a reservoir in order to meet a required output water supply. The operational rules are used in management of reservoirs. In applying engineering methods, the assumption is implicitly made that the same water input series will occur during the actual operation of the reservoir, and therefore, no risk is involved in meeting the specified demands, which is not correct.

The *experimental methods* are based on the generation of new samples of inputs and outputs, by using either operational rules or optimization techniques. Several attempts have been made to apply the optimization techniques in everyday decision making in reservoir storage operation. A relatively small number of cases can be cited for which the real decision making has been based only on these techniques.

The *analytical methods* have been developed mainly by probabilists, and some by engineers. To treat the storage problems mathematically many simplications on input, output, and boundary and initial conditions of reservoirs must be introduced. Solutions to such simplified cases depart so much from reality that the results could not be introduced into practice.

In order to treat the problems of two, three or more reservoirs, several approaches have been studied. For each reservoir, the useful storage capacity is divided in slices or layers, and each time-unit input and output are considered as discrete time interval values. By using discretization of the useful volume of reservoirs, and of time series, the discrete optimization techniques have been introduced. The review by T. G. Roefs (1968, Reservoir Management: The State of the Art, report 320-3508, IBM, Wheaton, Maryland) on the state of the knowledge of application of different methods for solving water storage problems, in planning and operation of reservoirs, pointed out that for many of the new ideas used a bridge was still missing between the suggested theoretical methods and the applications.

To attempt a better transfer of knowledge from research on water storage design and operation, practical pilot studies are needed. The most promising approach for successful results will be in analyzing the simplest cases first, say by studying one-reservoir problems, and then passing to the two-reservoir problems, and finally to the multiple reservoir problems.

To study the potentials of both, the experimental and analytical methods in solving the water storage problems, conceived as the supplement for, or the replacement of the existing engineering methods of reservoir planning and operation, it will be beneficial to test them in the frame of the general three research topics: (1) Methodology for determining the objective functions; (2) Study of the properties of the operational rules approach to operations of reservoirs; and (3) Study of the concept of equivalent reservoir(s).

1.4 Pilot Reservoirs

The pilot single or multiple reservoir cases should belong to entirely different water resource systems, giving an opportunity to investigate the above topic in different environments, of different project characteristics and reservoir objectives from the legal and physical constraints view points.

The detailed objectives of using the pilot cases of reservoir systems are to test the developed methods in planning and operation of reservoirs, which may prove adequate either to amend or replace the existing engineering techniques of reservoir operation.

1.5 Objectives of Needed Research

The detailed objectives of needed research are:

(1) Development of a methodology consisting of policy criteria, principles, methods, techniques and procedures, for designing the objective functions for planning and operation of water storage reservoir systems.

(2) Introduction of time factor into the objective functions, to represent a continuous evolution of the objective functions, because of various changes with time that affect the operation of storage reservoirs.

(3) Tests of the new methodology, including the time evolution of objective functions, on cases of the pilot reservoir systems.

(4) Investigation of the historical and present state of operational rules and a classification of types of existing operational rules for reservoirs, by indicating their advantages and disadvantages, as well as the criteria and indices of measuring their performance and the corresponding objective functions.

(5) A search for the reasonable bridge between the objective functions and the resulting operational rules of reservoirs.

(6) Comparison of the results obtained by existing operational rules with the results obtained by various methods and algorithms of mathematical optimization in reservoir operations.

(7) An investigation of how the existing operational rules may be improved, theoretically or practically, and by using advantages that the optimization and computer techniques provide.

(8) A generalization of the concept of operational rules, as applied to reservoirs, by using the available analytical methods or any other method suitable to accomplish a good degree of generalization.

(9) Tests of the results on the above objectives, (4) through (8), on pilot reservoir cases, as indicated in (3).

(10) Investigation of the concept of equivalent reservoirs as an alternative to the multi-reservoir multi-state operational schemes and algorithms.

(11) Design of a methodology consisting of criteria, principles, methods, techniques, constraints and procedures for the application of the concept of equivalent reservoirs, and determination of equivalent reservoir characteristics.

(12) Tests of results under the objectives (10) and (11) on pilot cases of reservoir systems.

PART II

PRESENT RESEARCH ACTIVITIES

2.1 Short-term, Real-time Control of Linked Reservoirs under Risk and Uncertainty (Prepared by J. W. Labadie)

Research activities have been concentrated on automatic control (using 10 to 15 minutes control time intervals) of stormwater detention reservoirs and adjustable gates on large urban stormwater conveyance structures. This work can be extended also to short-term operation of large multi-purpose reservoir systems. An important contribution in this work has been the blending of fully dynamic unsteady channel routing into the linked reservoir system optimization. A new dynamic programming approach has been developed for this purpose. In addition, the attempt is made to analyze the effects of errors in inflow forecast on the accuracy of optimal control policies.

A computer package has been developed for stormwater storage regulation in an urban environment, which combines a real-time adaptive forecast model, an unsteady flow channel routing model, and a dynamic programming optimization algorithm.

2.2 Optimal Multi-purpose Operation of Reservoirs in a River Basin (Prepared by J. W. Labadie)

In this work, the focus is on long-term management questions (i.e., 20 to 30 year runs of monthly time intervals related to the effective multi-purpose use of existing and planned reservoirs in a river basin.) The beneficial uses included in the case studies so far are water supply, recreation and energy development, though others can also be considered. Here, the efficient network optimization algorithms have been used (i.e., the out-of-kilter method), modified for the appropriate consideration of evaporation and channel losses, as well as stream-aquifer interaction. An important aspect of the ongoing research activities has been a realistic incorporating of existing legal and institutional structures that govern water allocation, into the model so that optimal policies are implementable within the political realities. This aspect is often ignored in reservoir systems analysis. Also, the computer program is developed to be conversational and fully interactive so that water planners and operators can use the model without having to be computer experts.

2.3 Optimal Location and Sizing of New Reservoirs and for Future Better Operation (Prepared by J. W. Labadie)

Activities are currently directed at looking at water resources development questions related to increased water requirements for energy development in the western United States. Attempts are being made to synthesize an algorithm which can optimally locate and size planned reservoirs in a river basin, in order to meet projected demands and a better operation, subject to water quality constraints and instream uses. The current approach is to link the existing river basin simulation models to an efficient search algorithm. Efficient methods for multiobjective trade-off analyses will be applied here since there are many conflicting objectives.

2.4 Optimal Control of Multi-Level Reservoir Inlet Structures for Water Quality Management (Prepared by J. W. Labadie)

The ongoing research work, conjunctively with the U.S. Army Corps of Engineers, is on applying the optimization methods to this problem of multi-level reservoir inlet structures. The current algorithm gives the daily gate-control policies for controlling certain water quality parameters in reservoir releases. Multi-level inlet structures are an effective way of taking advantage of stratification conditions in large reservoirs. Multi-dimensional dynamic programming has been used in this work.

2.5 Objective Functions for Reservoir Operations (Prepared by John T. Westgate, reviewed by V. Yevjevich)

To assess how well the current operating practices are performing, it is necessary to determine the objective function that is used for the reservoir operation. This function should reflect the objectives, the level of assurance of meeting objectives, and the trade-offs between the conflicting objectives, applied by the operating agency in making the real-time operating decisions.

An objective function is user specific. Since one looks at the real-time operation rather than at the project planning and design, decisions made at the time of project authorization are taken as given inputs. The political process which evaluates the projects and which distributes the benefits and costs is not the process one looks to optimize. Rather, once the political proc ss has assigned the benefits, costs and objectives, it is necessary to assess how well the operating rules and procedures fulfill these objectives.

Two pilot reservoirs have been chosen as the test cases for this investigation of objective functions. The Bonny Reservoir is located on the South Fork of the Republican River in Eastern Colorado, constructed in 1948-51 as part of the Missouri River Basin Projects. The reservoir's purposes include flood control, silt control, irrigation, recreation and fish and wildlife enhancement. While the part of a large river basin project, Bonny is a multi-purpose, single reservoir system from an operational viewpoint. Of particular interest is the underdevelopment in the authorized purpose of irrigation and the well-developed, unallocated purposes of recreation, fish and wildlife enhancement (Walleye breeding and migratory bird refuge).

The second pilot reservoir system is the Colorado Big Thompson Project. This Bureau of Reclamation Project, completed in 1959, is a multi-reservoir, multi-purpose project. Purposes include irrigation, municipal and industrial water supply, hydropower and recreation. This complex system of reservoirs, power plants, pumping stations, and trans-mountain tunnels, on both sides of the continental divide, should have an objective function that, while complex, will be more explicit in its trade-offs between power and water supply objectives, and the level of assurance versus the quantity objectives.

The research activities follow the purposes that have been determined by the enabling legislation project design reports, operating manuals, and as results of discussions with the Bureau operations' personnel. Connectives between system state variables (storage level, inflow, demand, date, release) and their impact on project purposes are to be found. These connectives may be qualitative, but quantitative estimates will be sought. Priorities, constraints, trade-off functions, and levels of assurance, will be looked for in the project documents, Bureau operation manuals, and from interviews with the Bureau operations' personnel. The end result of this investigation is expected to be a methodology for determining the objective functions in general, and for determining them for each pilot reservoir system, which should be a close approximation of those that are used by the system operators.

The methodology on the objective functions will display objectives, connectives, assumptions, constraints, trade-offs and the level of assurance, which will give it its final form of converting the system state variables into the measure of success of operation. This methodology on objective functions will be used in the study of the existing operation rules, and to test the alternative approaches to the operation methods.

2.6 Operational Reservoir Equations (Prepared by Lars Anderberg, reviewed by V. Yevjevich)

A reservoir release policy is based on information available on the system, and the release policy can never be better than the quality of this information. The key problem in reservoir operation is the estimation of future states of the system. The approach to reservoir operation in the present research activities is a blend of three basic lines: theory, methods and techniques, and practical engineering.

The theory of reservoir operation is relatively limited in its potential to application at present. Several areas are well covered in almost every aspect of reservoir operation, but the results need yet to be translated into practice. The fundamental concept in all decision making in reservoir operation is the risk. What impact may a decision made now have in the future? To evaluate such a risk, one has to look into the future and estimate the states of the reservoir system. This immediately brings up the question of how far into the future does one have to look?

Assurance of water and power supplies, with the associated risk, should be defined in a manner that satisfies the objectives of the reservoir system. A significant discrepancy exists often between the methods available and the techniques currently used in the real-time reservoir operation. The most prevalent method in reservoir operation is the rule curves, usually based on physical characteristics of reservoirs and plants, historical data on inflow and required outflow, but particularly on the experience with operations. The relative rigidity of these methods is corrected by the flexibility in the use of the operators' experience, as long as the reservoir system is not very complex. Since the reservoir systems usually grow bigger and more complex, the factor of experience in reservoir operation is expected to decrease with time.

The major research activities are directed to practical engineering, namely to the transfer of technology, with the theory and operational methods intended for the real-time reservoir operation. To accomplish the transfer of knowlege, the project is based on a teamwork composed of the personnel of operational offices of the Bureau of Reclamation and the Corps of Engineers, and the Colorado State University researchers.

The basic research goals are a development, through the stages of various optimization and other activities, of a practical methodology of how the operational equations (that should replace the operational rules) will be obtained for the given purposes of reservoir operation at a given time, information available, various constraints, the availability to forecast inputs and outputs, and similar state variables on which the decisions depend on how much water will be released in the next time interval. These equations will be valid as long as there is no basic change in objectives, constraints and state variables. Instead of using an optimization algorithm every time the decision for the release must be made, the operational equations are the condensed decisions for the many time intervals in the future. Simple plugging of the state and/or forecast variables into the operational equations produces the decision variable values. As soon as the general conditions start to evolve, the new operational equations should be recomputed.

2.7 Equivalent Reservoirs (Prepared by Ricardo Smith, reviewed by V. Yevjevich)

The application of operational rules to a complex water resources system, such as a multi-reservoir system, by using the mathematical programming is limited by the available computer memory and by the computer cost. The complexity is not only due to a large number of system components, such as reservoirs, plants and channels, but also that the system must satisfy several purposes. The requirements for computer memory and time are an exponential function of the number of system state and decision variables. Several approaches may be used to reduce the dimensionality of complex water systems. A commonly used approach is decomposition. The complex water system is decomposed into subsystems, individually simple to treat, allowing the recomposition of that system. This approach decreases dimensionality with respect to the original system. The recomposition is followed by an iterative approach guaranteeing that the output of a subsystem represents the proper input of the subsequent subsystem. Although this approach may reduce dimensionality of the original system, the computer time required may be such to make this approach in many cases uneconomical.

Another commonly used approach for decision making in complex water resources systems is to proceed with a mathematical simplification of the essential characteristics of the system, and in such a way as to allow the use of certain mathematical programming techniques. The problem is that the simplifications of the real system sometimes will solve a problem that is basically different from the real problem.

An alternative approach for the solution of complex water systems is called here Equivalent System Approach. This approach was used previously by some authors for certain particular cases. Certain restrictions made it difficult for the generalization. W. Hall (System Analysis for Water Resources Engineering, Class Notes, 1979) called it Optimal State Dynamic Programming, for solving a complex water resources system with the hydroelectric power being the major concern. K. Takeuchi (Optimal Control of Multi-Unit Inter-Basin Water Resources Systems, Ph.D. Thesis, University of North Carolina, 1972) called it Spatial Operational Rule, and used it to solve a system where water supply was the major concern. Mohamed called it Total Energy in Storage, and used it for hydroelectric power as the major concern. Bradford used a procedure, developed by Aoki, called Aggregation Procedure, that was applied to the design of a complex sewer system.

Basically, what this technique systems suggests is to reduce the dimensionality of a complex water resources system by defining such an equivalent system that represents in a certain way the original system, but that has less dimensions. For example, for the case of a multi-reservoir system, the equivalent reservoir can be a single reservoir with a capacity that will produce the same results in water release, hydroelectric power, flood control, etc., as would be obtained from the initial water resources system. For an equivalent system defined, its operational rule can be determined by using a convenient mathematical programming technique. The operational rule decision for an equivalent system is then re-allocated to individual reservoirs by following a certain pre-established procedure.

The previous research results in this "equivalent system" approach are limited in application for certain cases, because simplifications assumed by authors in the conceptualization of the equivalent system, in the reallocation rules for simple decisions, and in some cases in the suggested procedure dealing with optima that are not feasible, require an iterative method in finding the feasible solutions. The equivalent system approach needs criteria for defining the equivalence for various purposes under various conditions, for multiple purposes, for analysis and integration of constraints, for storage capacities, inputs, outputs, losses, production, etc., for decision-reallocation rules, for analysis of their feasibility and optimality and practicality, etc. are only some subjects that need and are being investigated in these research activities. The use of the approach of Equivalent System in combination with other techniques, such as decomposition-recomposition in solving the complex water systems, have been suggested as an alternative. Basically, the choice is between aggregating subsystems into an equivalent system, with its simple decisions (in dimensionality) re-distributed to subsystems by an appropriate method, or using the decomposition of a system into its many subsystems, with each subsystem decision coordinated or corrected for.

Fort Collins, Colorado
May 15, 1979

OVERVIEW OF WATER MANAGEMENT METHODS FOR THE TVA-OPERATED RESERVOIR SYSTEM

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Present Operating Methods

The reservoir system of the Tennessee Valley Authority is shown in Figure 1. It was designed to regulate the main river and major tributaries in the Tennessee Valley for the primary purposes of navigation on the main river and flood control on the main river and its tributaries. As much as consistent with these purposes it was to be operated for the production of hydropower. Over the years additional uses of stream flow regulation have become important. Today, the system is also operated for providing municipal and industrial water supplies, regulating flows to minimize the effects of effluents including condenser cooling water from TVA's thermal power plants, fluctuating water levels for the control of mosquitos and aquatic weeds, controlling levels and flows to improve fish habitats and for various recreational uses.

All TVA reservoir system operations are coordinated by the River Management Branch of the Division of Water Resources. Close day-to-day cooperation is maintained with TVA's Office of Power which is responsible for the economic use of the hydropower resources. TVA directs the operation of 27 TVA-owned hydroprojects and six Alcoa projects (425 MW). The total hydro capacity of the TVA-operated system is 3600 MW. In addition, TVA directs the power operations of eight U.S. Army Corps of Engineers hydro projects in the neighboring Cumberland River system (853 MW). These hydropower resources are scheduled together with 13 thermal power plants (23 000 MW) to meet the TVA power system load.

The scheduling of the reservoir system is guided by:

1. Normal maximum and normal minimum pool elevations
2. Flood control guides and regulating zones
3. Normal ranges of pool levels
4. Balancing of storage volumes between reservoirs
5. Economy rule curves and a basic rule curve
6. Power demands, and
7. Hydrologic conditions.

Normal maximum and minimum pool elevations serve as guides within the operating space defined by the absolute constraints. Absolute contraints are project characteristics or minimum requirements specified by project design. Examples of these are top of gates, minimum navigation depths, minimum turbine intake levels, maximum turbine capacity, etc. They cannot be violated without impairing project safety or a specified design purpose. The guides introduce additional, but less stringent limitations on operation and are met in a certain order of priority to achieve the mandatory operation purposes specified in the TVA Act. Such limits are the normal maximum and minimum pool elevations, flood control guides and flood regulating zones, minimum hydropower reserve (basic rule curve), minimum recreation levels, minimum flow and others. The guides (also called "user supplied constraints" in contrast to the absolute constraints) may be departed from if it is considered desirable to trade off increments of one achievement level in favor of others. Such trade-offs may occur between flood control and power generation, fish and wildlife enhancement and power generation, recreation and power generation, etc.

The economy rule curves serve as guides for power system operation planning. They represent values of energy in storage as function of storage and time of the year. They are used by the System Loading Branch to schedule hydro requirements for estimated power loads and steam unit schedules.

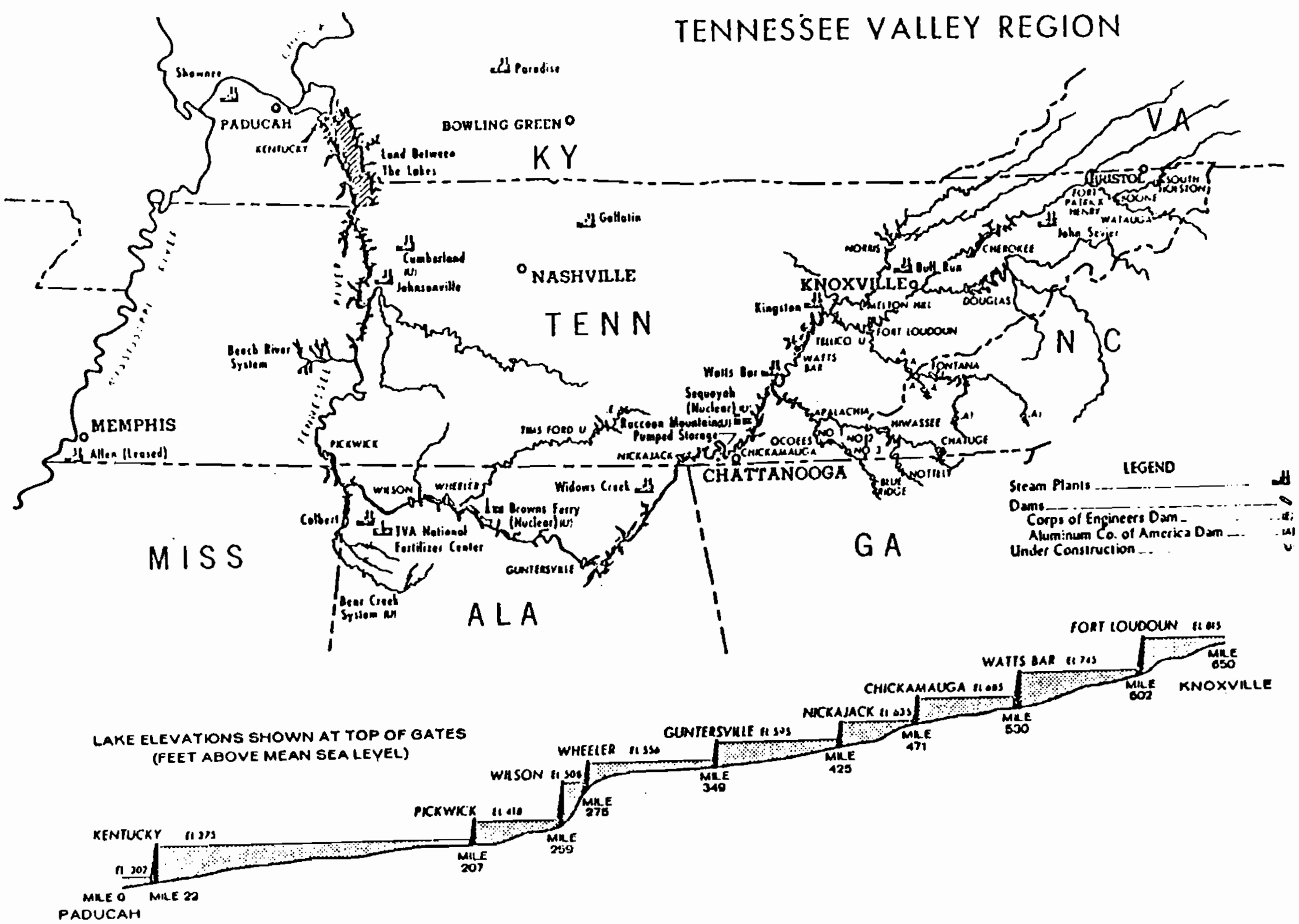


Figure 1 - TVA-Operated Reservoir System

The predicted hydrological conditions, the hydro requirements and the normal pool elevation guides are used in determining the daily reservoir headwater elevations and releases. Except for occasional adjustments, these guides have remained unchanged over the years.

The reservoir system is operated on an annual cycle. The flood season usually begins in December. The reservoirs are drawn to low levels to provide flood control space in the multipurpose reservoirs and to minimize spill in the single purpose (power) reservoirs. The reservoirs are allowed to rise gradually until April and then more rapidly to reach full pool at the end of the flood season (around the middle of April). Starting in late spring or early summer (June or July), the water is drawn out gradually to supplement natural flow for navigation, power production and water supply. Drawdown becomes more rapid during the generally drier fall months. This lowers the reservoirs for controlling the next season's runoff. All reservoirs are returned to essentially the same low levels at the beginning of each flood season. The ratio of total useful storage to annual basin runoff is about 0.3.

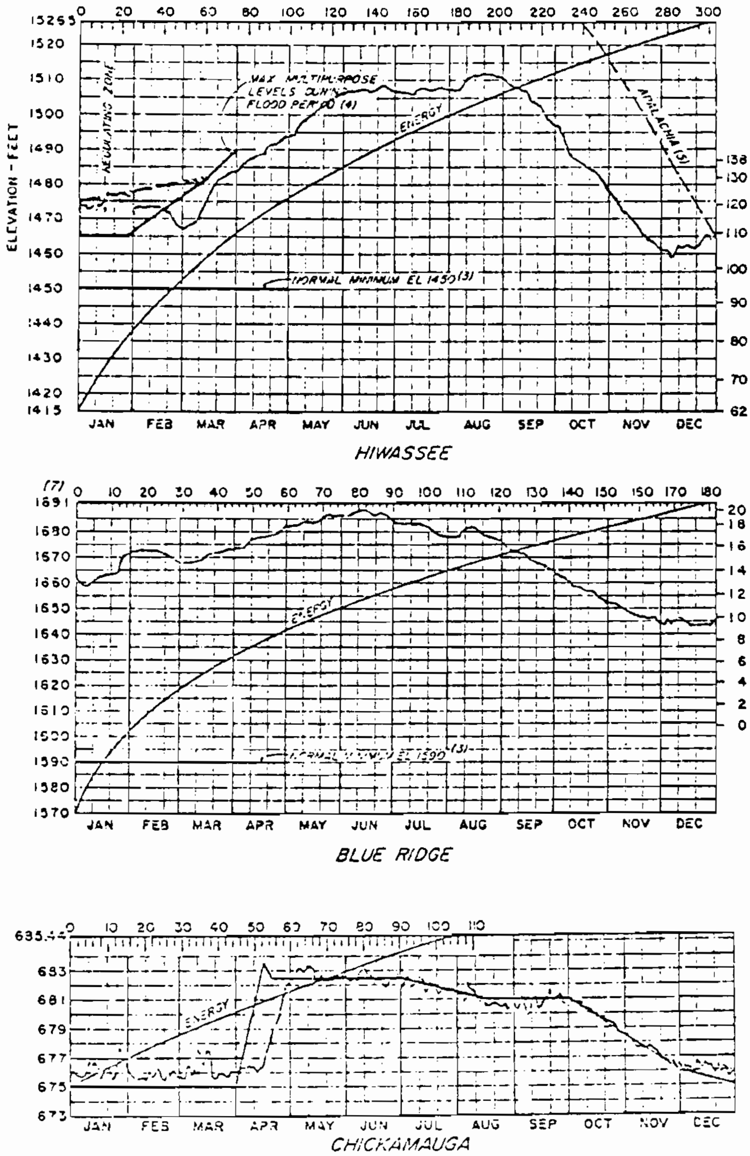
Examples of seasonal water levels for a tributary multipurpose reservoir, a tributary power reservoir and a Tennessee River multipurpose reservoir are shown in Figure 2. An annual operation summary based on monthly reports of the River Management Branch is shown in Figure 3. It indicates for each project and twelve months of the year the reported operation purpose. As shown by this overview, the most frequently reported operation for tributary multipurpose reservoirs is aiding to meet high system loads by peaking. Tennessee River multipurpose reservoirs are operated to follow seasonal guides.

Development of Enhanced Water Management Methods

The manager of a large multipurpose reservoir system is confronted with the daily task of determining reservoir levels and releases that best fulfill the specified purposes now and over the long range. He must have at his disposal estimates of inputs and demands for the upcoming decision period and certain computational procedures by which he can assess the consequences of a contemplated operation. Such an assessment must give due consideration to the inputs and demands of the near and distant future and to reservoir system capabilities in order to achieve continuous satisfactory performance of the reservoir system. For a large multipurpose system this is a rather complex task because of the interaction of the many physical system components, the competing economical, environmental and social factors and the uncertainty of future inputs and demands. Presently no methods exist to fully deal with this problem.

In response to this situation, TVA in 1971 started a project to enhance presently used water resource management methods. This project is aimed at augmenting information on inputs and demands and at evaluating this information by fast and comprehensive computational methods. As a result, the system manager can compare several operation alternatives in terms of their near and distant future effects on reservoir operation goals. To the extent possible, quantitative measures of effectiveness will be provided to assess their relative merits.

The project includes the development of mathematical models for the various aspects of the operation problem. They comprise forecasting methods for system inputs and demands, simulation models for the physical character-



Multiple Purpose Tributary Reservoir

Single Purpose Power Reservoir

Multiple Purpose Main River Reservoir

Figure 2 Annual Operation of TVA Reservoirs (1977)

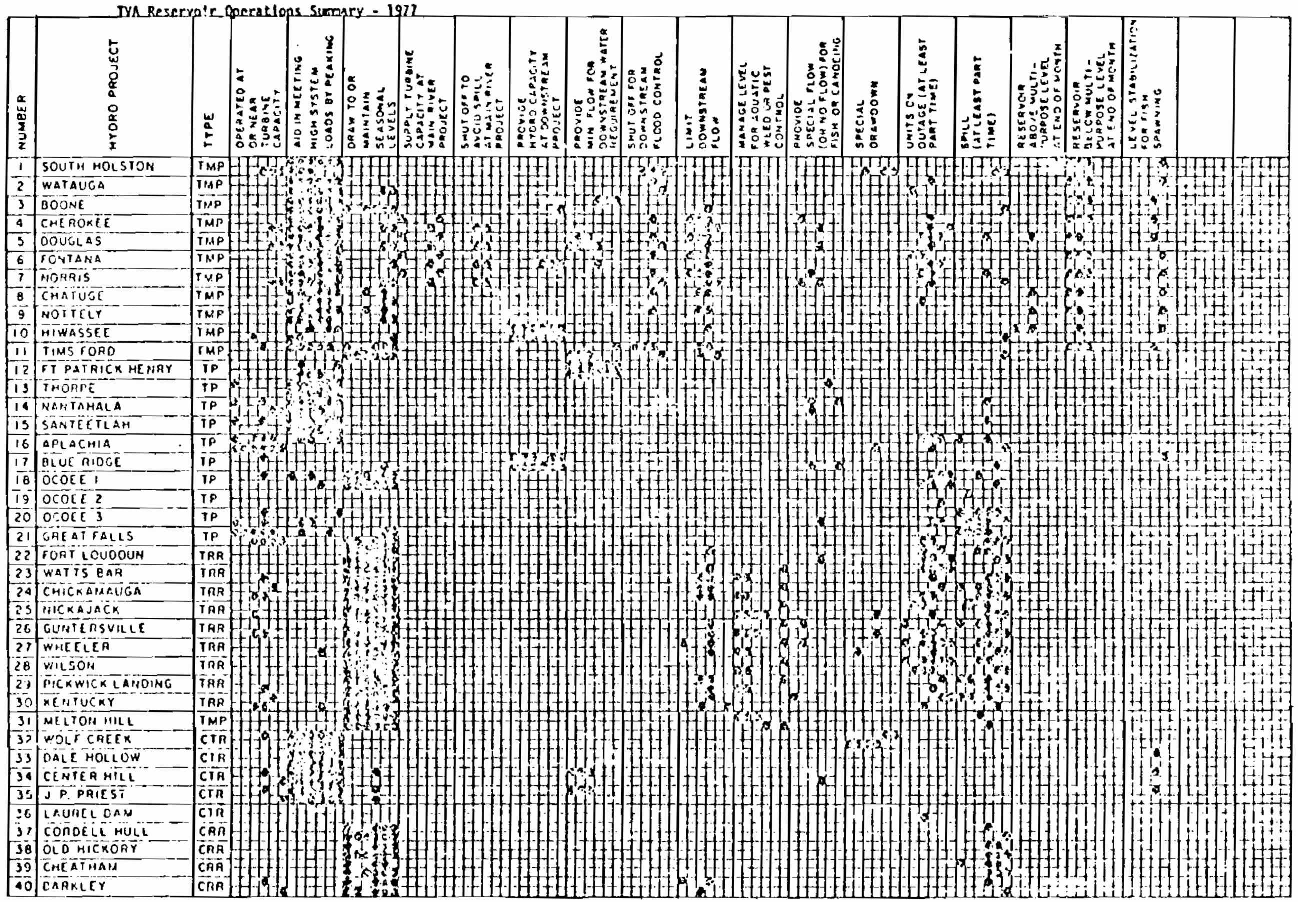


Figure 3 Operations Summary of TVA-Operated Reservoirs (1977)

istics and processes of the reservoir system, methods for evaluating operating objectives, and reservoir scheduling models. The combined use of these methods will aid in finding operation policies which best approximate or meet the objectives of system operation. These methods, besides their use in day-to-day applications, can also be used in the systematic analysis of planned changes of the system or its operating policies.

This section is limited to the discussion of one category of models to be developed by the project, that is reservoir scheduling models. Two different types of models are under development: long-range models using weekly time steps over planning horizons of up to one year or more and short-range models using daily or hourly time steps over planning horizons from one day to two weeks. An overview of planned developments (including the completed model 1) is given in Table 1. The models include capabilities to reasonably represent the physical processes going on in the system. They include objective functions which relate economic measures to decision variables, such as flows and storage. Also, they include algorithms which can satisfy all absolute and user-supplied constraints, simultaneously or in order of specified priority. Within the feasible solution space, these algorithms can find preferred solutions which minimize a specified performance criterion, such as a system performance index which can be a composite cost of several operating purposes or a single purpose operating cost, such as power generation cost.

The principal benefit of the project will be for the reservoir manager to have a capability on hand for fast and comprehensive evaluation of operation alternatives. This will provide more knowledge about the possible outcomes of contemplated operations for the near and for the distant future. It is fair to assume that this increased knowledge will have a beneficial effect on the outcome of the operational decision process. The project is expected to reduce the amount of manual computations and to replace these by faster and more accurate computer programs. However, more work than at present will be necessary in preparing input data, running computer programs and analyzing output. Interactive computer use will permit fast and comprehensive display of results.

Any computational scheme that simulates a complex real world problem is affected by the limitations on mathematical modeling of the physical system and by the uncertainties associated with the inputs and demands. Therefore, evaluation and implementation of the results still depend on the judgment of the user, on how he interprets the augmented but still incomplete information provided by the methods. Thus, the final result depends on the decision of the system manager. The proposed methods can only provide guidance leading to that decision.

Effect of System Configuration on Operation

The watershed of the Tennessee River is 40 910 square miles (106 000 km^2). The mean annual discharge at its mouth is about 66 400 ft^3/s (1 900 m^3/s). The average annual precipitation over the drainage basin is 52 inches (1 300 mm), ranging from 38 inches (in 1941) to 65 inches (in 1973). The total useful controlled storage of all major hydroprojects is 13.7 Mio acre feet ($17 \cdot 10^9$ m^3). The average annual air temperature in the region is about 17°C, with a weekly average low of 4°C in January and a high of 26°C in August.

Table 1 - Water Resource Management Methods
Scheduling Models Under Development

No.	Name and Description	Use and User	Status
1	Weekly Reservoir System Planning Model		
	End-of-week storages in 19 reservoirs and weekly average discharges of 42 hydroprojects and other weekly information on the reservoir system is calculated using dynamic programing by successive approximations (DPSA). Planning horizons can be 52 weeks or less. Includes objective functions for five operating objectives which relate system operating costs to decision variables (discharges and/or water levels). Minimizes a system performance index which is expressed as weighted sum of these costs over the planning horizon. Inputs are power system characteristics, loads, streamflow and air temperature sequences for the planning horizon.	For systems analysis studies by Water Management Methods Staff in cooperation with the River Management Branch, Power System Operations, Power Supply Planning and other TVA organizations interested in finding multiple purpose reservoir operation policies. Also to be used in weekly operations planning by RMB to evaluate operations for projected inflows and power loads over planning horizons of ±12 weeks.	Has been applied to planning studies such as maintaining high summer levels. Documentation available (Report WM28-1-500-11).
2	Weekly Scheduling Model		
	A linear programing model is used to evaluate for one week at a time the schedule that minimizes expected power generation cost for forecasted inflows and loads after successively satisfying all constraints by preemptive priority ranking. Long range system operating guides in terms of expected power costs and other indicators are computed on a weekly basis for time horizons of 52 weeks by a separate program which uses stochastic dynamic programing (STORES) on a dimensionally reduced reservoir system.	To be used by the Water Management Methods Staff in cooperation with the River Management Branch for week by week scheduling of the reservoir system. Also used for projecting reservoir system performance. For planning studies in cooperation with other TVA organizations.	In advanced stage of development. Documentation in preparation.

Scheduling Models Under Development - continued

No.	Name and Description	Use and User	Status
3	Daily Reservoir Scheduling Model This model will be used to determine end of day headwater elevations and daily average discharges, which meet or approximate specified targets and/or minimize a multipurpose performance index subject to all operating constraints. The model will be developed first for the main river projects, then expanded to include the tributary reservoirs.	To be used in support of daily scheduling now performed by manual iterative methods by the River Management Branch.	In early stage of development.
4	Hourly Reservoir Scheduling Model A short time step flow routing method in combination with a scheduling algorithm will be used to find headwater elevations and discharges for reservoirs upstream from flood sites. Constraints will be satisfied in order of priority. A single flood control objective or a combined flood control and power objective will be optimized. Planning horizons are from a few days to two weeks.	To be used by the River Management Branch in pre-flood, flood and post-flood situations. Used repetitively for each new streamflow forecast, once and more often per day.	Planned. Components for reservoir routing are in various stages of development.
5	Hourly Subsystem Scheduling Models Mathematical programing methods will be used to calculate headwater elevations, releases, power generation and other information of interest subject to multipurpose constraints and performance criteria. Models will be limited to subsystems in which short time variations of flows and water levels cause concern, such as in areas of steam plant cooling water use or pumped storage operations.	To be used by the River Management Branch to find schedules which are in accordance with multiple purpose operation or to adjust proposed schedules to improve subsystem performance.	Planned. Pilot studies for steam plant cooling water use and pumped storage operations have been made. Limited documentation available.

A schematic arrangement of the reservoir system is shown in Figure 4. The major storage reservoirs (Cherokee, Douglas, Norris, Fontana and Hiwassee) are situated parallel to the series of nine main river reservoirs. The cumulative distribution of flow, detention capacity and installed hydrocapacity along the main river is shown in Figure 5. It represents percentages of the system totals upstream of a given river mile. The figure shows that about 60 percent of the detention capacity is upstream from Chickamauga Dam and thus available for the protection of Chattanooga, the location with the largest urban damage potential in the system. Thirty percent of the detention capacity is in Kentucky Reservoir at the downstream end of the system and only 10 percent is available on a 300-mile stretch of the main river between Chickamauga and Pickwick. Hence, flood water can either be retained in tributary storage upstream from Chattanooga or in Kentucky Reservoir at the downstream end of the system. This latter reservoir serves mainly for flood control on the lower Ohio and Mississippi Rivers. Once water is released from the tributaries it can only be passed on through the system. Water travel time from the tributary dams to the chain of main river reservoirs is about half a day. From the confluence (origin of the Tennessee River at Knoxville) to Chickamauga Dam, the travel of a wave takes about half a day. Across the 300-mile stretch from Chickamauga to Pickwick wave travel time is about one day and through Kentucky Reservoir it takes another half a day. Hence, about 2½ days are required for a water wave to travel from the tributaries to Kentucky Dam. Heavy precipitation events may occur in sequence as closely as three days in a row. Therefore, quick decisions to relieve the system of accumulated flood storage are necessary.

The distribution of installed hydro capacity along the main river is also shown in Figure 5. About 2000 MW or 55 percent of the total hydro capacity is installed in the nine main river projects. Their share of total hydro production is of the same order, 52 percent in 1977 and 63 percent in 1978. Annual hydro productions and average hydrocapacity use rates for the TVA system are shown in Table 2.

The specific storage distribution in the system is reflected in the weekly models. These models only treat 19 tributary storage reservoirs as state variables. The other reservoirs, including all main river reservoirs, are assumed to follow fixed level guides. However, this assumption is dropped in the daily and hourly models when all reservoirs have temporary storage capacity.

Operation Under Varying Hydrologic Conditions

Flood Period

The flood season in the Tennessee River basin is distinctly limited to about 5 months of the year lasting from about the middle of December to the middle of April. Of 132 floods on record that would have gone beyond flood stage in Chattanooga, some 116 including all major ones occurred during this period. Most of the remainder occurred on the fringes of this period, including May and November, and some smaller floods occurred scattered over the period from June through October, see Figure 6. This rather distinct pattern allows most of the flood storage space to be allocated to other operating purposes outside the flood season.

The distinctly seasonal pattern of flood occurrence does not relieve the system operator of the difficult task of short-range planning for flood

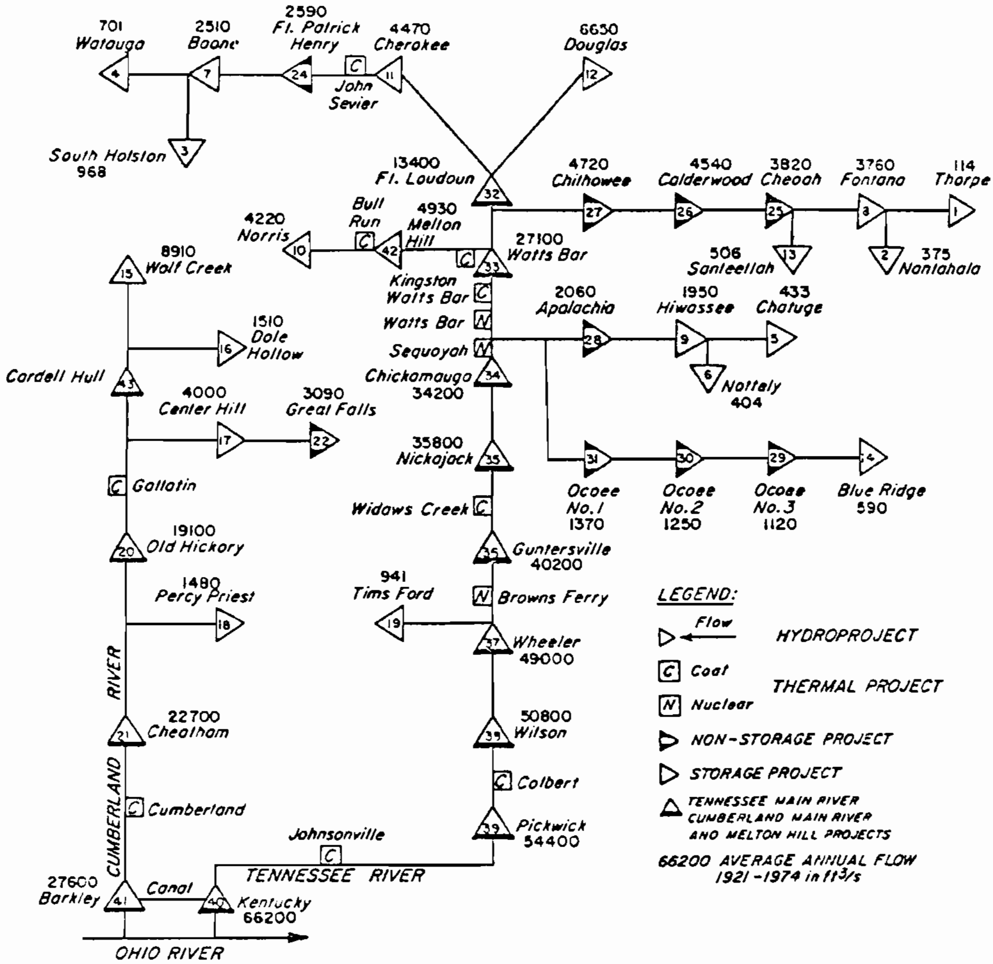


Figure 4 Schematic Arrangement of TVA Reservoir System
(Cumberland River included)

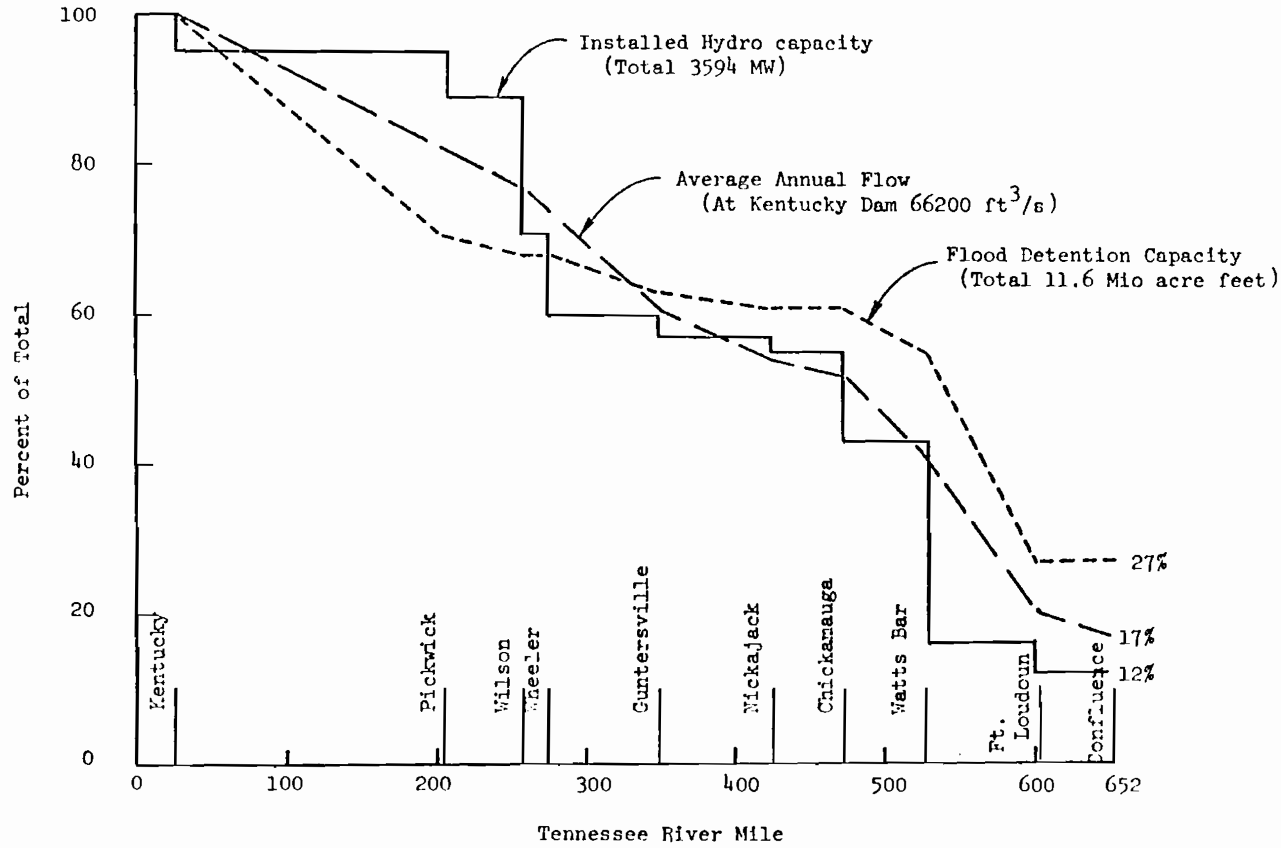


Figure 5 Distribution of System Characteristics

Table 2 Tennessee River Basin Hydrocharacteristics

Year	TVA Hydro Capacity* MW	TVA Hydro Generation*† TWh	Overall Use Rate÷ %	Annual Flow$ $10^3 ft^3/s$
1966	3468	12.677	42	49.8
67	3387	15.032	51	72.9
68	3497	17.282	56	50.3
69	3511	13.287	43	55.3
70	3530	13.955	45	56.4
71	3536	14.376	46	65.8
72	3579	17.881	57	81.8
73	3585	20.555	65	99.3
74	3597	19.717	63	90.3
75	3604	19.403	61	97.5
76	3632	16.465	52	59.7
77	3648	16.101	51	80.9
78	3766	17.323	53	
	3565	16.466	50	66.4

Note: Increases in capacity are due to unit modifications and rehabilitation.

* Tennessee River basin only. Alcoa (424 MW) included, but Great Falls (32 MW) excluded.

÷ Given for Fiscal Year (July–June until 1976; 15 months until Sept. in 1977 and from Oct. to Sept. starting 1978).

$ at Kentucky Dam for Calendar Year (40 200 square miles); adjusted for storage.
Units: 1TWh = 10^{12}Wh = 10^9KWh

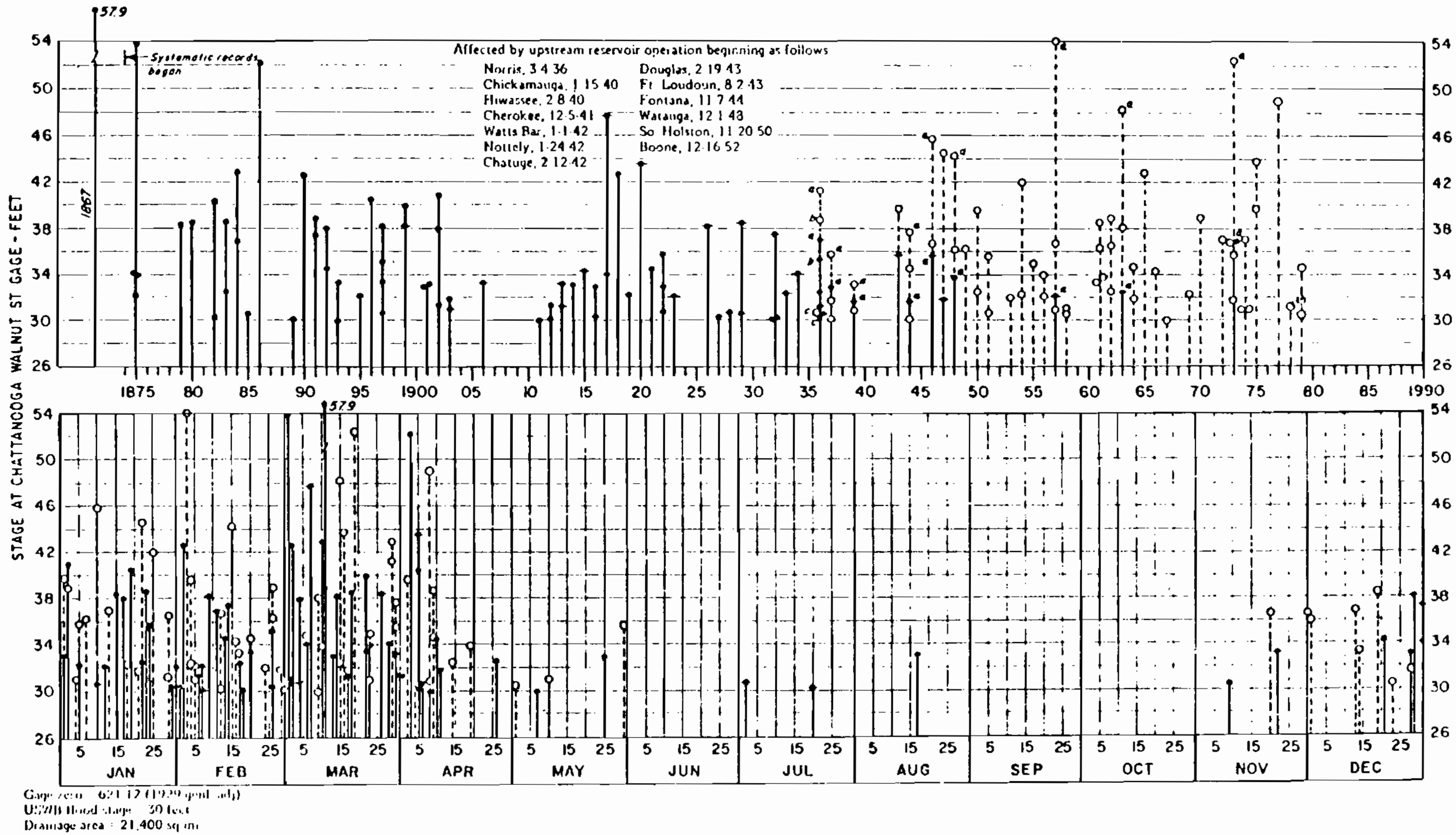


Figure 6 Flood Stages at Chattanooga

control within the flood season. Measures taken in anticipation of major flooding include passing of early flood waters as rapidly as possible through the system and at reducing or shutting off the tributary releases which could contribute to the flood crest at Chattanooga. In this latter case, runoff is temporarily stored in the flood detention space of the tributary reservoirs. When the downstream flood crest has passed, the tributaries are returned to seasonal levels to recover the detention space for future floods. Maximum turbine capacity, sometimes supplemented by spill, is used in this post-flood drawdown.

Storage space reserved for flood detention at various times of the year is shown in Table 3. It amounts to about five inches throughout the flood season and one inch during the summer months. The total precipitation of 80 percent of all major precipitation events was equal to or less than five inches and almost all these events occurred within two to seven days.

Transition Period

A transition period follows at the end of the flood season during which the reservoirs can be filled to summer levels. Rapidly decreasing flows after April make it desirable to fill the reservoirs to summer levels by the end of April. The volumes to be filled between March 31 and April 30 are shown in Table 4. The table also shows the cumulative detention volumes expressed in average monthly flow and various levels of natural flows during April at selected dam sites. A comparison of required and available flows indicates that even with all water retained, the guide levels cannot be reached 50 percent and more of the time in many reservoirs. Therefore, it is desirable to store as much water as possible toward the end of the flood season. This is also desirable with respect to water quality and fish habitat management. Late reservoir filling causes a reduced cold water storage which may make it impossible to provide tailwater temperatures below 20°C later in the year for cold water fish habitat maintenance.

Normal Operation

The mean monthly rainfall and runoff for the Tennessee River Basin is shown in Figure 7. More than half of the total runoff occurs during the first four months of the year. Due to the relatively limited storage capacity of the reservoir system, most of this runoff must be passed on so that flood control space can be retained. The total system storage/flow ratio is 0.3. The storage/flow ratios of major tributary reservoirs range from 0.69 to 0.21, as shown in Table 5. The storage/flow ratios of the main river reservoirs range from 0.08 to 0.01. These ratios are indicative of the limited amount of storage available for regulation.

Figure 7 also shows the effect of TVA streamflow regulation on the monthly flow regime of the Tennessee River at Kentucky Dam. The overall change in flow regime is small with reductions of about 10 percent of monthly average flow during the high flow period (January through March) and increases of up to 25 percent during the low flow period (June through November). The total shift of flow from the high flow period to the low flow period is little more than one inch of water over the drainage basin. This is less than 15 percent of 6.4 inch, the already limited total useful storage.

Table 3 Flood Detention Capacity at Various Times of the Year

Time	Above Chattanooga		Above Kentucky Dam	
	$10^9 ft^3$	inch*	$10^9 ft^3$	inch**
January 1	277.7	5.6	506.8	5.4
March 15	224.2	4.5	450.8	4.8
Summer	52.7	1.0	113.2	1.2

* drainage basin at Chattanooga is ~21 400 square miles
** drainage basin at Kentucky Dam is 40 200 square miles

TABLE 4 Allowable Reduction of Flood Detention Capacity in April

Project	Flood Detention Capacity Change March 31 - April 30 1000(ft 3/s)day	Cumulative Detention Capacity (ft 3/s) month	APRIL FLOW Maximum (ft 3/s) month	APRIL FLOW Median (ft 3/s) month	APRIL FLOW Minimum (ft 3/s) month
S. Holston	28.6	950	3 000	1 480	410
Watauga	11.4	380	1 890	1 010	321
Boone	12.0	1 730†	6 810	3 920	975
Cherokee	276.3	10 940†	13 800	6 790	1 770
Douglas	372.6	12 420	19 000	8 820	3 990
Fontana	220.3	7 340	11 600	5 210	2 580
Norris	205.4	6 850	17 500	5 870	1 540
Chatuge	22.4	750	1 390	607	252
Nottely	27.7	920	1 410	568	243
Hiwassee	88.3	4 610†	6 750	2 760	1 250
Tims Ford	33.8	1 130	3 340	1 330	408
Ft. Loudoun	40.8	24 720†	38 800	16 700	6 710
Watts Bar	107.9	42 510†	87 100	38 000	13 700
Chickamauga	119.0	51 090†	102 000	48 200	17 900
Guntersville*	69.7	53 410†	138 000	59 500	20 200
Wheeler	166.4	60 090†	169 000	73 900	26 100
Pickwick*	144.3	64 900†	189 000	84 400	27 600
Kentucky	362.0	76 960†	213 000	102 000	35 200

*Guntersville includes Nickajack; Pickwick includes Wilson.

†Includes detention capacity change of all upstream projects.

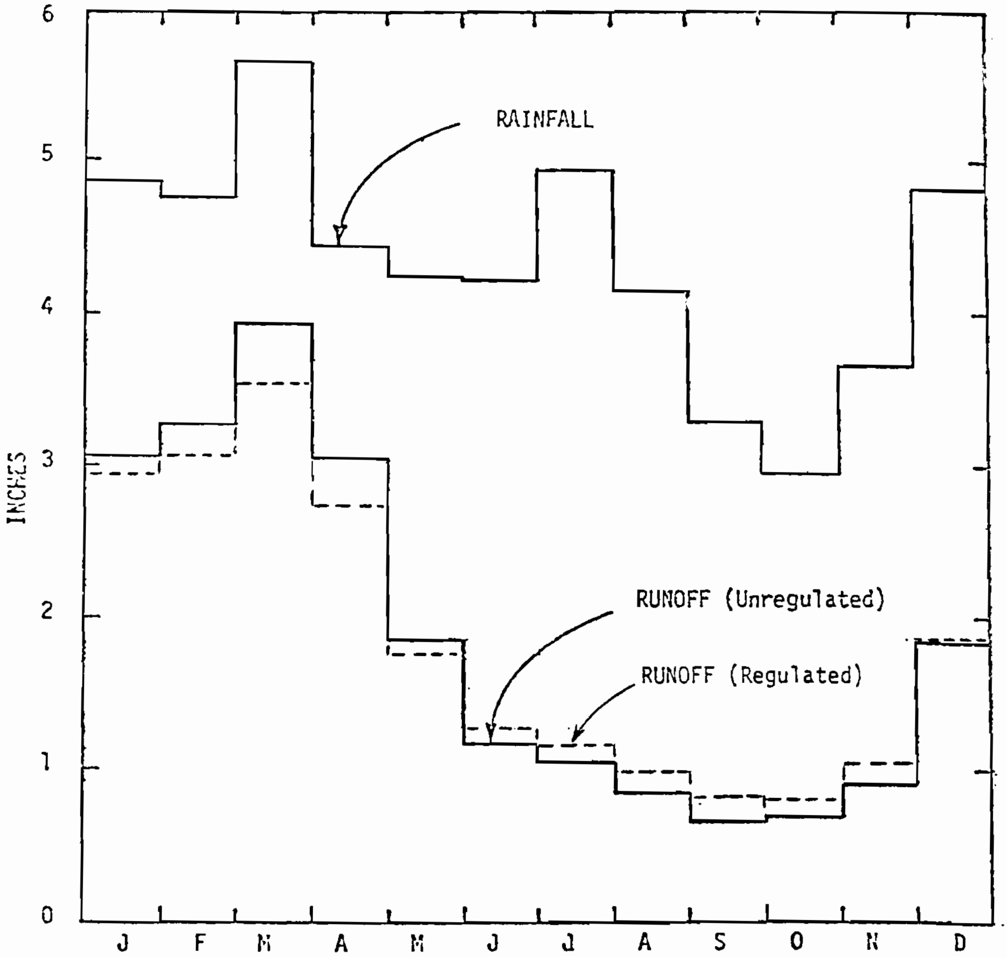


Figure 7 MEAN MONTHLY RAINFALL AND RUNOFF FOR THE TENNESSEE RIVER BASIN

Table 5 Storage Characteristics of Major TVA Reservoirs

Project Type	Project Name	Useful Storage $10^9 ft^3$	Mean Annual Flow ft^3/s	Storage/ Flow Ratio
Tributary Multipurpose	S. Holston	19.1	977	0.62
	Watauga	15.4	707	0.69
	Norris	83.7	4230	0.63
	Cherokee	50.1	4490	0.35
	Douglas	54.5	6710	0.26
	Fontana	41.2	3790	0.34
	Hiwassee	13.3	1970	0.21
Tennessee River Multipurpose	Watts Bar	16.5	27400	0.02
	Chickamauga	15.1	34400	0.01
	Wheeler	15.3	49500	0.01
	Kentucky	174.6	66900	0.08

Low Flow Period

The historic record shows that flows decrease rapidly after April and continue to decrease or stay low throughout October. Flows are usually augmented by regulation over a period of six months from June through November, as shown in Figure 7. For operation planning it is of interest to know how low the flow will get and how long low flow periods may last. An example of historic flow occurrences over the past 75 years is shown in Figure 8. It represents histograms of quarterly average flows for a major tributary basin (8 900 km^2). While the distribution is almost symmetric around the mean in the first quarter, there is a definite increase in the frequency of low flows in the remaining quarters of the year. Examples of synthesized low flow periods are shown in Table 6. Assuming independence between the quarters, the most critical period lasting three months with an average flow of 500 ft^3/s starts in summer with a frequency of 2/75; a six-month period with an average of 1000 ft^3/s also starts in summer with a frequency of 1/75; a nine-month period with an average flow of 1500 ft^3/s starts in spring with a frequency of 1/75. Such flows and durations have actually occurred in the 75-year record. At present, the historic record has not been fully analyzed for its information contents with respect to the beginning, duration and severity of droughts.

It is not possible to recognize, let alone to predict, the beginning of a drought or any other of its characteristics. A drought is usually recognized when we are in the middle of it. But even at this stage, synthesis of possible flow sequences and the use of the proposed models will help to operate through such periods.

Concluding Remarks

This overview is concluded by pointing out some critical areas where enhancement of presently used methods will most likely result in improved reservoir management.

- The amount of flood control space to be filled at the end of the flood period has an important effect on water quantity and quality management later in the year. Enhanced weekly scheduling methods will be especially helpful during the transition period (March to May) to increase as much as possible the amount of water available for later multipurpose use.

- Enhanced flood routing methods will result in more precise predictions of flood crests at potential damage sites and of flood wave travel through the system. Improved quantitative precipitation forecasts will help to more effectively prepare the reservoir for flood control. Enhanced daily and hourly scheduling methods for tributary and main river reservoirs will produce schedules that minimize flood damage at minimum loss to other purposes (power).

- Long-range planning for low flow periods can be assisted by synthesizing critical flow sequences for several months into the future. The scheduling methods can be used with these inputs to continually update storage requirements for streamflow augmentation, energy in storage, water quality, fish habitat and recreation needs.

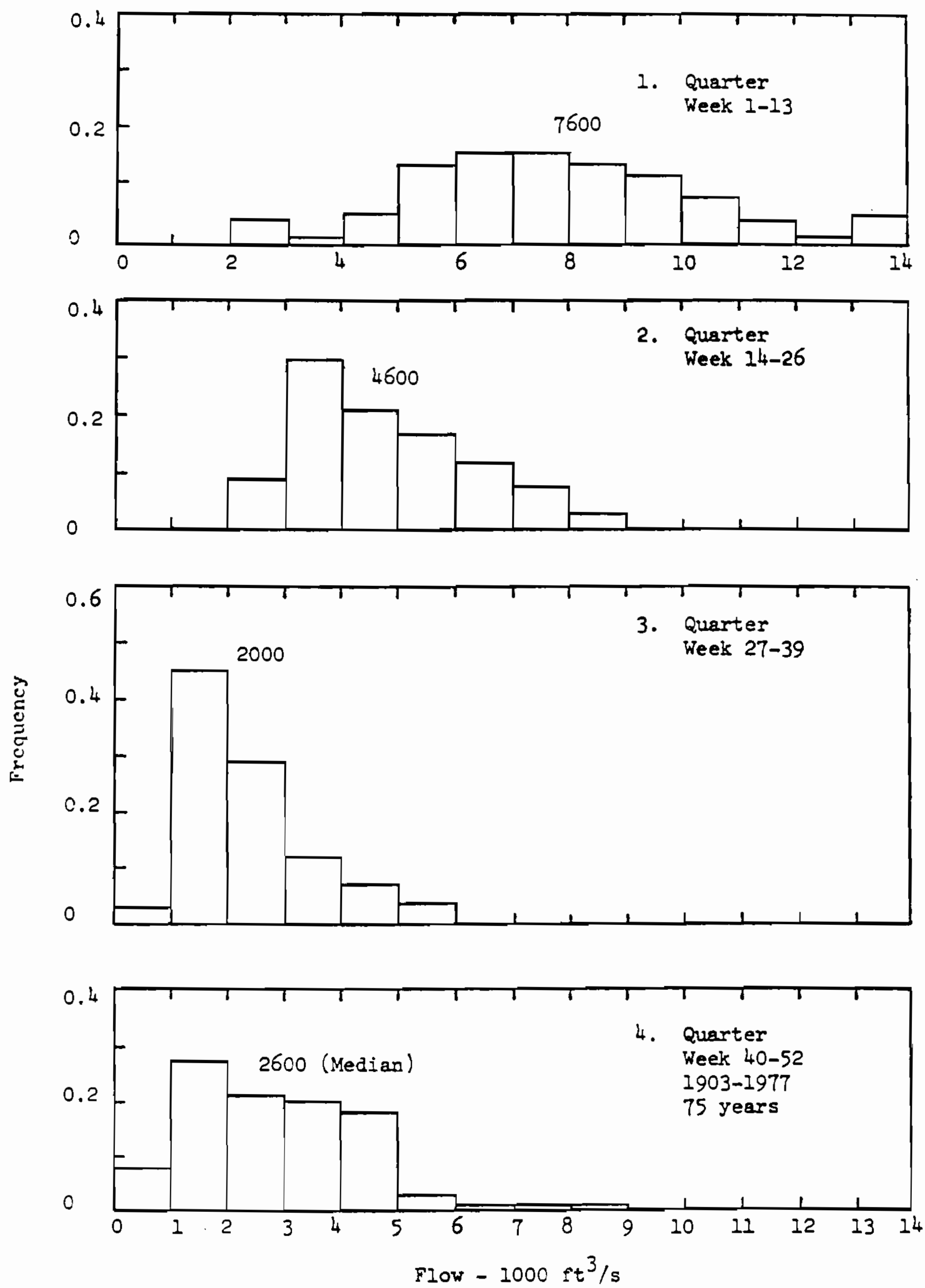


Figure 8 - Cherokee Dam - Average Quarterly Flows

TABLE 6 Frequency of Low Flow Periods in the Holston River Basin (Cherokee, 3428 square miles or 8875 km^2)

Duration Quarters of the Calendar Year 1	2	3	4	1	2	3	Months	Average Flow ft 3/s	Flow Classes During Quarters 1000 ft 3/s	Average Low Flow ft 3/s	Frequency
*							3	7970	2/3†	~2500	3/75
*	*						6	6430	2/3,2/3	2500	1/75
*	*	*					9	5010	2/3,2/3,0/1	1800	1/450
*	*	*	*				12	4490	2/3,2/3,0/1,1/2	1750	1/450
	*						3	4880	2/3	~2500	6/75
	*	*					6	3530	2/3,0/1	1500	1/75
	*	*	*				9	3330	2/3,0/1,1/2	1500	1/75
	*	*	*	*			12	4490	2/3,0/1,1/2,2/3	1750	1/750
		*					3	2170	0/1	500	2/75
		*	*				6	2560	0/1,1/2	1000	2/75
		*	*	*			9	4340	0/1,1/2,2/3	1500	2/750
		*	*	*	*		12	4490	0/1,1/2,2/3,1/2	1500	2/2250
			*				3	2950	0/1	500	6/75
			*	*			6	5430	0/1,2/3	1500	1/75
			*	*	*		9	5250	0/1,2/3,2/3	1800	1/225
			*	*	*	*	12	4490	0/1,2/3,2/3,0/1	1500	1/1350

† 2/3 means that flow during this quarter of the year is between 2000 and 3000 ft 3/s, or about 2500 ft 3/s.

- The enhanced methods will be useful planning tools to evaluate the effects of operation policy changes on multipurpose reservoir operation. A case in point would be the testing of a modified flood control approach that requires less storage reservation.

The operation of a reservoir system is rather uniquely determined by its physical characteristics, the hydrologic regime of the basin and the primary purposes it is designed to serve. Therefore, it is generally not possible to transfer methods wholesale frome one system to the other. Also, computational efficiency of computer programs requires that they be tailored to the specific task on hand. But general concepts are not bound by these limitations. In this sense it is hoped that some of what has been said is useful to others interested in reservoir system operations.

ON THE SHORT-TERM CONTROL OF MULTIOBJECTIVE RESERVOIR SYSTEMS: A CASE STUDY OF THE KAPOS BASIN, HUNGARY

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Abstract

Problems associated with the general formulation of multiobjective dynamic reservoir system control are discussed. A simple three-level hierarchic structure is introduced to overcome difficulties of imbeding policies and related to uncertainties. Lower level models are nested into higher level models. Real-time control aspects are also considered. An example for the Kapos River Basin, Hungary, is presented.

1. Introduction

The optimal short-term operation of multiobjective reservoir systems has nowadays become a world-wide concern both for academic circles and for practitioners. Decision makers need tools to operate their reservoir system in an optimum, or rather, in the best manner. On the other hand it is still a challenging task for researchers to find out optimal control strategies for multiobjective systems. There is quite an amount of literature devoted to this subject (for water resources problems see Haimes et al.,1974). However, in the past few years static cases, i.e. multiobjective design, were mainly attacked. The reason for this probably lies in the fact that for operational control one has also to know the dynamics of the system in question. So, beyond the preferences, inequality type constraints etc., one has to know explicitly what the dynamics of the elements of water resources systems are. It means that the dynamics is also a constraining factor.

2. Short-term multiobjective control: in general

The control problem for a multiobjective dynamic system can be formulated a follows: Find a control vector $\underline{u}$ which simultaneously minimizes n objective functions, i.e.

$$\min_{\underline{u}} \left\{ f_1(\underline{u}, \underline{x}), \ldots f_n(\underline{u}, \underline{x}) \right\} \tag{1}$$

where f_i is the ith objective function of the system and $\underline{x}$ is the vector of state variables representing the behavior of the system. Obviously there are constraints imposed on the control functions; those which fulfill these constraints are said to be admissible:

$$\underline{u} \in U \tag{2}$$

Obviously there are constraints on the state variables too. These constraints are given by the state equation

$$\frac{d\underline{x}(t)}{dt} = \emptyset\ (\underline{x}(t),\ \underline{u}(t),\ t) \tag{3}$$

which represents the dynamic behavior of the system and by other constraints imposed on the states, i.e.

$$\underline{x} \in X \tag{4}$$

Considering the ith objective the related objective function is generally in the form of

$$f_i\ (\underline{x},\underline{u}) = \psi_i\ (\underline{x}(t_f)) + \int_{t_o}^{t_f} r_i\ (\underline{x}(t),\ \underline{u}(t),\ t)dt \tag{5}$$

where t_f is the terminal time of the control, ψ_i and r_i are certain cost functions. The cost function ψ_i expresses the importance of reaching a given terminal state while r_i measures the costs associated with the tracking of a given trajectory as well as with the control effort. The above control problem consists of

- tracking control
- terminal control
- minimal energy control.

In the objective vector (1) each function is expressed in the form of (5). Generally there is no solution which simultaneously minimizes the objective vector. (Necessary and sufficient conditions for the existence of a unique optimal control, that is called superior solution are discussed in Goodwin et al., 1975.) what is to be found, however, is the set of noninferior, or Pareto optimal, solutions. To find the noninferior solutions for this control problem is extremely difficult even for deterministic systems. (Some solution methodologies are discussed in Salukvadze, 1974). The problem becomes more involved when uncertainties both in the state of the system and in the preference structure unavoidable in water resources systems, are also to be considered. In this case the expected value of the objective vector is to be minimized, i.e.

$$\underset{\underline{u}}{\text{MIN}}\ E\left\{ \underline{f}\,(\underline{u},\ \underline{x}) \right\} \tag{6}$$

subject to constraints similar to Eqs. (2), (3), (4) but corrupted with some noise processes. In short-term reservoir operation a feed-back solution is sought in the form of

$$\underline{u}\,(t) = L\left[\left\{ \underline{x}\,(\tau)_{t_0}^{t} \right\} \right] \tag{7}$$

where L stands for the control law. In any feedback control predictions are of vital importance. Since in short-term forecasting/control problems uncertainties play essentially an important role the above approach is to be reformulated.

3. A hierarchic model structure for operational control

Below a model structure is outlined shortly which is of quite simple structure and yet gave sound results when applied. With this approach

- the generation of the entire noninferior set is avoided,
- a short-term policy can be imbedded into a longer one,
- the different uncertainties can explicitly be taken into account.

This model consists of three hierarchically connected sub-models as depicted in Fig.1.

The first level model preforms the function of generating a priority list for the objectives and preferences.

There are actually two possibilities here:

- Either to fix a preference ranking as a function of time for the objectives, say flood control, water supply, fishing, recreation etc. (This obviously is not a model *per se* but is entirely due to the subjective decision of the system operator and essentially is a lexicographic ordering amongst the objectives for a given part / season of the year);
- Or to set up a model for the long-term statistical behavior of the reservoir system concerning different resource/demand structures.
  In this way the ergodic states of the system can be derived (Zsuffa, 1979).

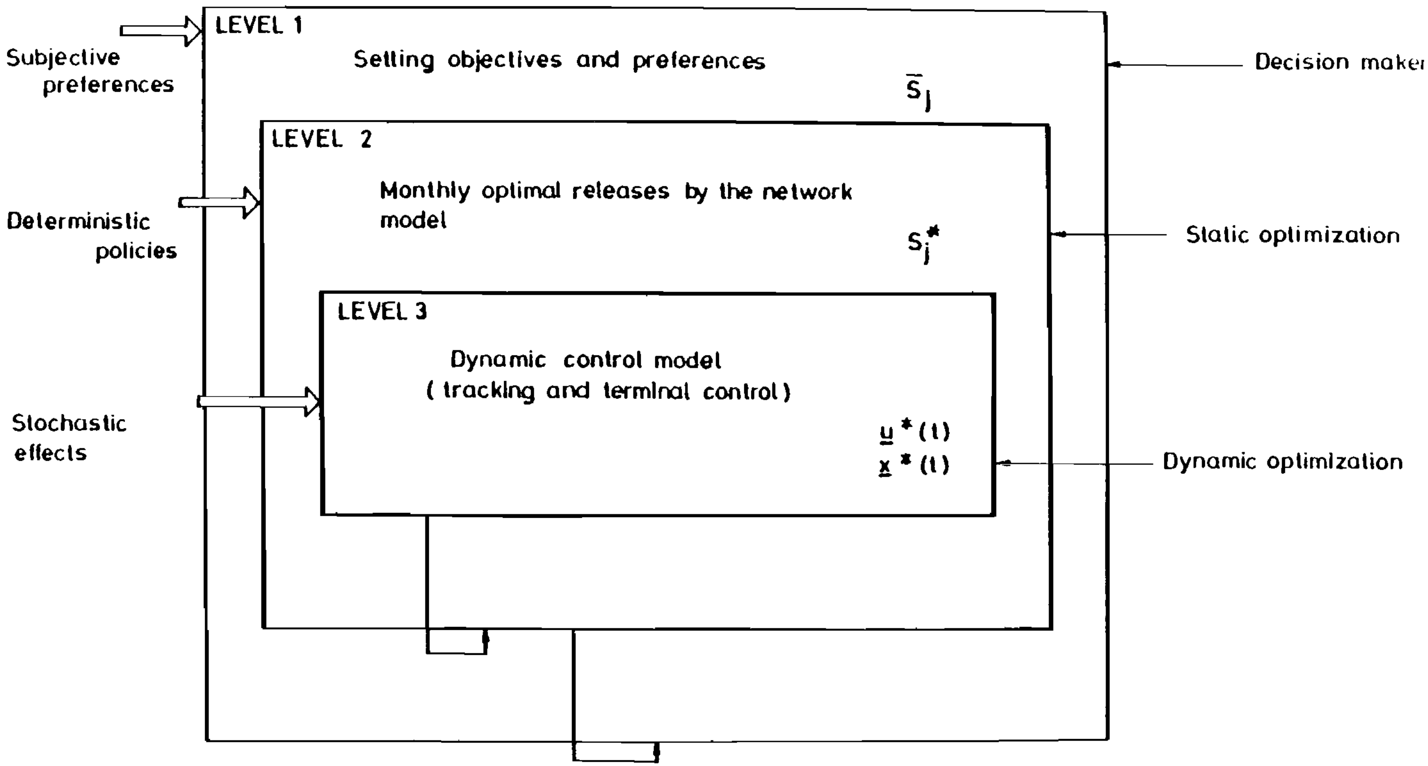

Fig 1. Conceptual scheme for the hierarchic control model

The second level model determines medium-term (5-days, 10-days, or monthly) control strategies. At this level a static optimization is carried out concerning the one-interval-ahead resource allocation. Local random disturbancies are not considered here, deterministic policies are derived. At this level the elements of a reservoir system, i.e. reservoirs and channels, are regarded as far as their cooperation is concerned. The model used utilizes a capacitated network formulation. Following Sigvaldason /1976/, the decision making process is being simulated which consists of two parts:

- determination of the medium-term strategies,
- monitoring the system response

which follow each other in a recursive manner. The objectives of the second level model are to match the "optimal-in-the-long-run" strategy (given by the rule curve) as well as to minimize the deviation between the actual and the so-called ideal state of the system. Obviously some, at least rough empirical prediction for the future inflows is required at this stage. Sigvaldason's zoning concept (op. cit) is applied here according to which the elements of the system are divided in to zones each of which corresponds to a particular usage/objective. The zones bounded though these lower and upper limits can also be changing in time. Different operation modes are considered then, such as :

- Inter-reservoir zonal operation,
- Relation between storage and flow violations,
- Inter-reservoir policies.

If a violation of the rule curve given by the level-1 model occurs then a penalty is assigned that is linear in a particular zone but is piece-wise linear for the whole region. With those penalties and flows into/from the zones a special linear(LP)model can be set up. The important thing here is that the multiobjective problem is reduced to a single objective optimization using a parameteric type approach. To ensure enough flexibility however, there is still some room for including refined and quantified preferences. This is done through the choice of penalty coefficients. Then total penalty is minimized. The solution of this medium-term optimization problem is obtained by the out-of-kilter algorithm (Ford and Fulkerson, 1961).

The third level model determines the short-term (one day or even real-time) control policies in such a way that it matches the control strategy given by the level-2 model and at the same time considers the random disturbances. This model is also discrete and linear but now dynamic. The short-term control model is based upon the state space description of the processes involved. The objective function is scalar-valued and is a quadratic form

$$\min_{\underline{u}} J_3 = E\left\{ \| \underline{x}(t_f) - \underline{d} \|^2_{Q_0} + \sum_{t=t_0}^{t_f-1} [\| \underline{x}(t) - \underline{d} \|^2_{Q_1} + \| \underline{u}(t) \|^2_{Q_2}] \right\} \quad (8)$$

where t_f is the terminal time of the control, $\underline{d}$ is the state trajectory given by the level-2 model, Q_0, Q_1 and Q_2 are, respectively, cost matrices. Similarly to Eq. (5) this objective function also expresses tracking control (to the desired state $\underline{d}$), terminal control and minimal energy control. The constraints of the control problem are given by

i) the dynamic state equation

$$\underline{x}(t+1) = \emptyset(t+1,t)\, \underline{x}(t) + \Gamma(t)\, \underline{u}(t) + \underline{w}(t), \quad (9)$$

where $\emptyset(\cdot)$ is the state transition matrix expressing the structural properties of the system, $\Gamma(\cdot)$ is the control transition matrix while $\underline{w}(\cdot)$ is a noise process called process disturbance

ii) the measurement equation

$$\underline{z}(t) = H(t)\ \underline{x}(t) + \underline{v}(t) \tag{10}$$

where $\underline{z}(t)$ is the vector of measurements concerning the state variables, $H(t)$ is the measurement matrix, while $\underline{v}(t)$ is again a noise process called measurement uncertainty;

iii) the statistics of the disturbances;

iv) the initial values for the states;

v) Eq(2), i.e. the controls should be admissible.

Here again a feedback control is sought in the form of Eq.(7). Due to the linearity of the system we can take advantage of the separation principle (Kalman, 1961) which states that the above stochastic control problem can be separated into two problems, viz. 1) the estimation of the state variables from noisy measurements and 2) deterministic dynamic programming performed on the estimated state variables. The optimal feedback control is

$$\underline{u}^{*}(t) = -L(t)\ [\hat{\underline{x}}\ (t/t) - \underline{d}] \tag{11}$$

where $L\ (\cdot)$ is the feedback control gain matrix, $\hat{\underline{x}}\ (\cdot)$ refers to the optimal state estimation given by a linear Kalman-filter.

4. Case Study: the Kapos Reservoir System

The above control methodology was used to determine optimal short-term control policies for the reservoirs in the Kapos Basin. The Kapos River lies in southern Hungary, its catchment area is 3210 sq km. There are 29 existing reservoirs in the basin and 12 additional reservoirs are being planned to be established in the future. The objectives of the existing reservoirs are as follows: flood protection, agricultural, industrial and communal water supply and recreation. The discussed methodology was used to find control policies for reservoirs above the city of Kaposvár. Figure 2 shows the configuration and the capacitated network formulation of the reservoirs. As an example Fig. 3 shows the ergodic states of Reservoir Toponár together with the state trajectories resulting from the medium and short-term optimal control. In this study two assumptions were made, namely it was assumed that for the medium-term control one-step-ahead runoff volumes are known at least with a 90 percent reliability and that an on-line measurement system is attached to the basin. In other words the prediction part of the control was simulated.

It can be seen from Fig.3 that level-3 control is forced to reach the state trajectory resulting from the level-2 control considering in the meantime the local uncertainties. This was achieved by assigning high elements to the terminal control cost matrix.

5. Conclusions

In this paper a simple hierarchic model structure was presented for the short-term control of multiobjective reservoirs. A priority list for the objectives and preferences is being given by the level-1 model. Level-2 model generates the medium-term policies using quantified preferences of the decision maker. Level-3 model determines the short-term dynamic control strategies.Here,stochastic effects are also considered. Models of lower level hierarchy are imbedded in higher level models. In such a way the multiobjective control problem is reduced to a series of single objective optimization problems.

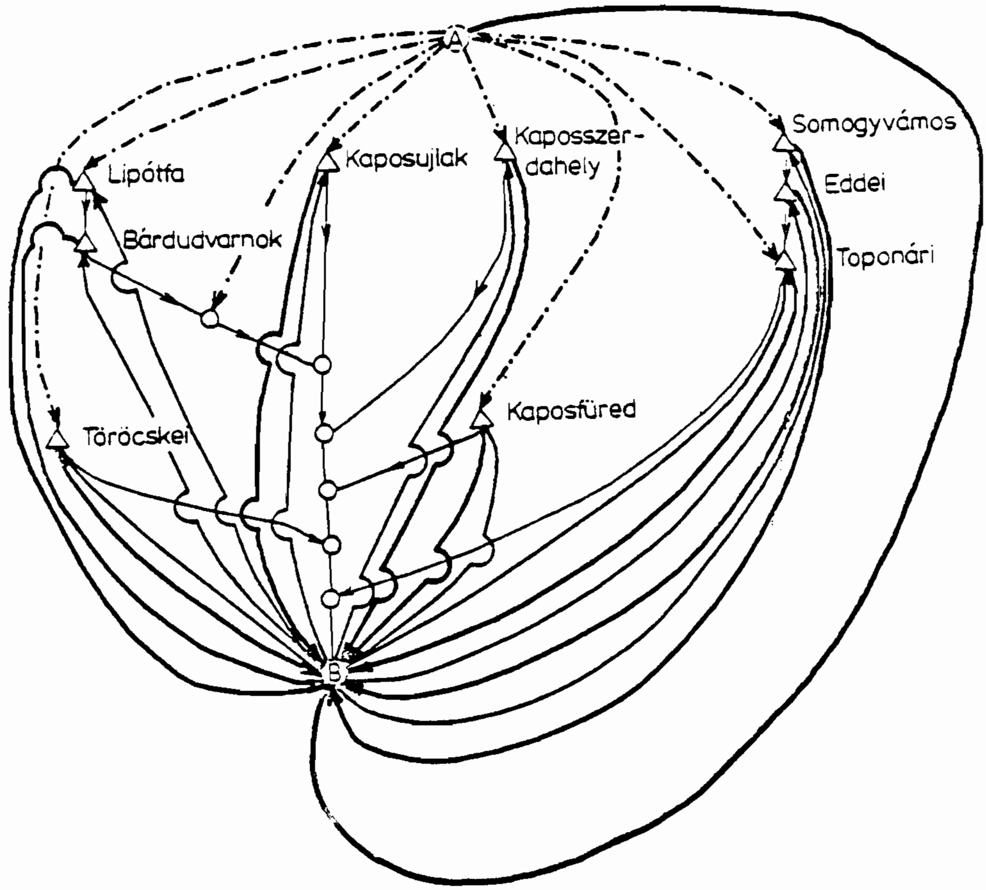

Fig. 2 Capacitated network for a Sub-basin of the Kapos Reservoir System.

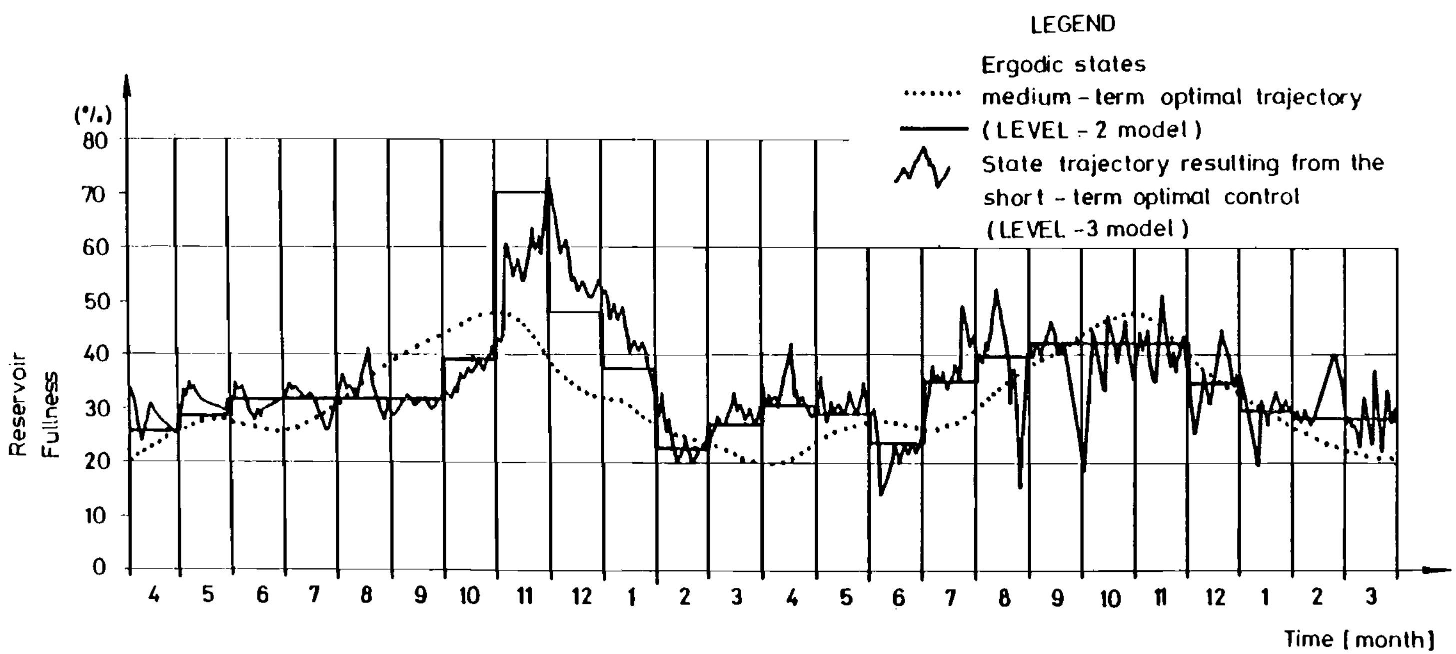

Fig. 3. Ergodic states for the Toponár Reservoir as a function of time, state trajectories for the medium and short - term control strategies

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THE ODRA RIVER WATER RESOURCE SYSTEM: A CASE STUDY

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1. INTRODUCTION

The most densely populated and most industrial area of Czechoslovakia is situated in the Odra River Basin. The mines, power stations, chemical and metallurgy industries are concentrated in the towns of this drainage basin. The problems of water resources management are the consequence of these conditions which can be characterized by the following data: the density of the population is nearly twofold and industrial production is three times greater than the national averages. On the other hand the natural water resources per unit of area form one third of the national average.

2. WATER RESOURCES

Water resources of the Odra River Basin are limited for two reasons. The first limiting factor is the hydrologic conditions (the basic hydrologic data area shown in Table 1), and the second reason is the poor water quality. The quality of water in the lower parts of the Odra tributaries and specially of the Odra River itself, prevents it from being used for demands created by the national economy.

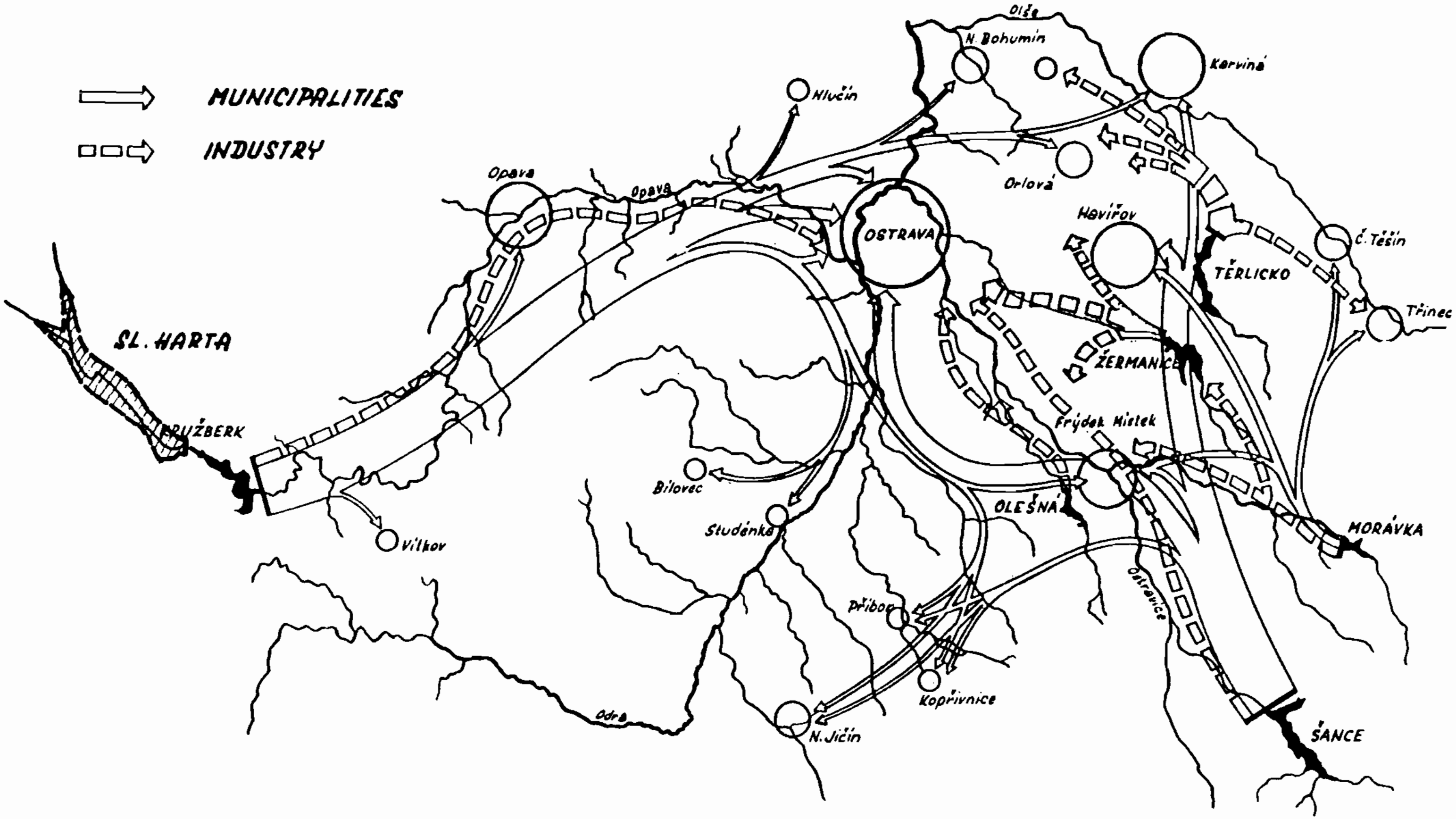


Figure 1. Odra River W.R.S. (Delivery of Water)

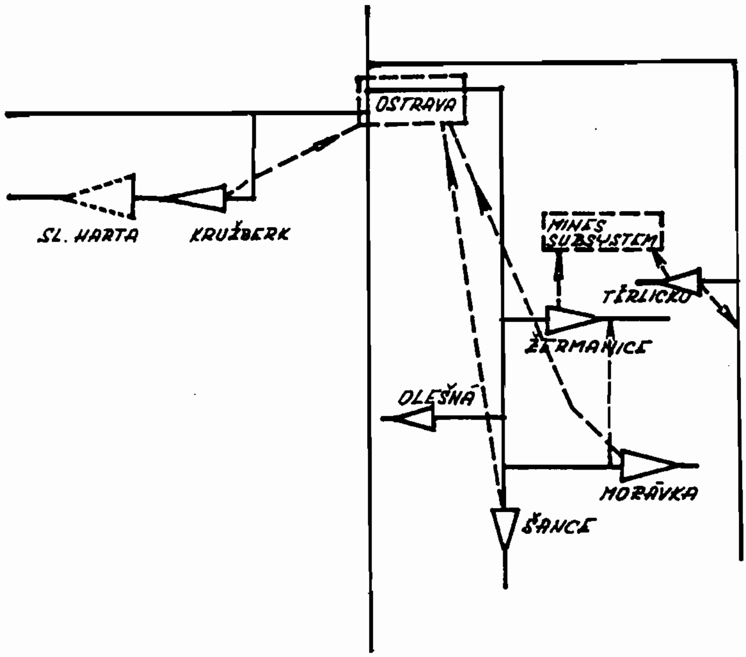


Figure 2. Graph of the Odra W.R.S.

Table 1. Hydrologic characteristics.

River	Catchment Area km^2	Mean Annual Flow m^3.s^{-1}	1-day Flow m^3.s^{-1}	Max 1% Flow m^3.s^{-1}
Opava	2089	15.0	2.03	600
Ostravice	827	14.2	1.24	1150
Olše	1120	12.5	1.00	700
Odra	5840	55.8	5.77	2300

3. "MULTIMODELLING"

The growth of water demands for municipal supply, for industry, for recreation and flood control made it necessary to design a water resource system, taking into account interrelationships among different system components. An optimal operation policy for the system was sought as the demands could not be met by the individual, isolated water sources. When the theoretically determined optimal operation policy did not make it possible to meet water requirements with a required reliability, the necessity to expand the system by additional storage facilities had to be considered.

The analysis of water resource system expansion and determination of the operation policy was done by the application of several different models in accordance with the precepts given below:

(a) systems analysis of the problem should determine the basic objectives of the system,

(b) different models compatible with these objectives should be developed,

(c) the methods used in the development of models should reflect the various premises, hypotheses and theories concerning the system,

(d) the water demands of the system should be transformed into the inputs of the models; in different models, the same demands should be used,

(e) the operation procedures developed by application of different models should be comparable.

The following set of models was investigated:

3.1 Chance-Constrained Model I

In the preliminary investigation, a chance-constrained model with a linear decision rule was used as it was developed for the analysis of multipurpose water resource systems (for description and application of the model see e.g., ReVelle, Joeres and Kirby 1969; Kos 1975). The linear decision rule of the chance-constrained model used for the system of reservoirs Šance, Morávka and Těrlicko was formulated as follows:

$$X_{i,k} = s_{i-1,k} + b_{i,k}$$

where $X_{i,k}$ are the releases in the month i and reservoir k; $s_{i-1,k}$ are storage values at the beginning of the month i (i.e., at the end of the previous month i-1) in reservoir k; and $b_{i,k}$ are the parameters to be determined by the model. This chance-constrained model was originally designed for minimization of reservoir capacities necessary for some multipurpose demands, with a given reliability.

In this case study, the formulation of the task was a little different. The capacities of the total storages were given and the maximum draft that could be delivered by reservoirs with a given probability was the main aim of investigation. Therefore, the problem was simplified, as the direct solution described by Eastman and ReVelle (1973) could be used.

The results were not encouraging, as the draft (for 98% reliability) was much lower than that used in current operation policy. There were several reasons for this result. In the chance-constrained model, the probabilities are given for each period (month) regardless of the other periods. The consequence of this assumption is that each month of the year would have low flows of the given probability of exceedance (reliability). This is not generally true, with the exception of a very high correlation between the values in the following months. This persistence of drought in subsequent periods exists but it does not last the whole year. The analysis of critical periods of the observed data has shown that with a reliability of 98%, five to seven months can be treated in this way.

There was another possibility to lessen the claim for such high reliability each month. Then with the monthly reliability of 90% (for the draft constraint) the carry-over reliability of approximately 98% was attained.

The application of the deterministic formulation of the model not for the really observed data but for a "representative sample" constructed by the methods of synthetic hydrology was another alternative possibility. In fact the probabilistic formulation of the chance-constrained model with the linear decision rule is based on the same principle but it takes no account of the relation of reliabilities among the months. The model with this representative sample t kes this relation into account and has further advantages. The original formulation of the chance-constrained model is suitable for the reservoirs with a one year cycle only. On the contrary, reservoirs with a carry-over potential can be treated by this representative sample model when the length of the cycle is two or three years. For longer periods, this method is not suitable. Therefore it was used for the current system which included the reservoir Šance with some carry-over potential, but it could not be used for the enlarged system with the Slezská Harta reservoir (see Table 2).

A further problem of the chance-constrained model application with the linear decision rule, was connected with the main purpose of the system. The linear decision rule is suitable for the case when water can be offered to the users according to the storage capacity of the system. The only user that is able to use the

Table 2. Reservoirs of the Odra River W.R.S.

Reservoir	River	Mean Annual Flow $m^3.s^{-1}$	Storage mil. m^3 Total	Active	Relative
Kružberk	Moravice	5.95	35.6	20.0	0.11
Šance	Ostravice	3.11	54.2	45.8	0.47
Morávka	Morávka	1.77	10.1	4.4	0.08
Žermanice	Lučina	1.15	20.1	18.5	0.51
with transfer of water		3.57			0.16
Těrlicko	Stonávka	1.14	24.3	22.0	0.61
Olešná	Olešná	0.52	4.3	3.0	0.18
Slezská	Harta Moravice	5.35	210.0	200.0	1.19

amount of water above the constant yield is the use (for environmental purposes), for improving the water quality by dilution. However, the water quality of the lower part of the Odra River is poor, and the effect of dilution is so small that it can hardly justify this aim. Therefore, the results of the chance-constrained Model I (i.e., the model with the linear decision rule) were considered as an informative tool and in the final decision analysis they had a lower degree of importance.

3.2 Chance-Constrained Model II

The chance-constrained model that can be used without the linear decision rule was derived by Curry, Helm and Clark (1973). This model is, however, more complicated and the calculation more time-consuming. The output of this model does not consist of the parameters b like in the chance-constrained Model I, but it is formed by the draft values.

In application to operation policy design, this model was a little cumbersome. It led to a linear program of a prohibitive size for the computer used in this case study when 40 years (i.e., 480 monthly periods) were used. Further, the same objection applied, as in the case of the chance-constrained Model I. The values of the releases were variable but the demand was nearly constant. Therefore this model had to be modified according to the conditions of the Odra River water resource system.

The system of the reservoir Slezská Harta was analyzed initially by the method of synergism with the deterministic simulation model (see below). This simulation was used for the determination of the stochastic variation of the draft of the reservoir Slezská Harta. The overall demand of the system was constant but the draft of individual reservoirs differed according to the method of synergism. As the reservoir Slezska Harta is the main reservoir of the expanded system, the maximization of the constant part of the draft produced by this reservoir was the principal task of this model: therefore, the chance-constrained model with stochastic demands and with maximum constant draft was used. Then the formulae for the model were simplified and the values were calculated from the following equations (Kos and Zeman 1976):

$$S = \max_{n} \left(\frac{1}{r^n} \left(\frac{1-E^n}{1-E} . X - g_n (1-p) \right) \right)$$

where S is the storage necessary for the constant part X of the draft with given reliability p; E is the coefficient of monthly losses (e.g. for 1% losses E = 0.99); and g_n is the quantile of the convolution distribution of Z_n where

$$Z_n = \sum_{j=1}^{n} E^{n-j} . (I_j - D_j)$$

where I_j is the inflow in period j and D_j is the stochastic part of the draft (demand of the system cooperation).

For given values E and p, the storage S is a function of X: S - f(X). As the inversion function $X = f^{-1}(S)$ cannot be expressed analytically, a numerical solution of this equation was used.

3.3 Simulation Models

The deterministic simulation model using the observed monthly flows (the period 1931-1970) was the basic model used for the following purposes:

(a) for determining the claims of the system on the stochastic part of the Slezská Harta draft (input for the model described in 3.2),

(b) for verification of the results (namely of the operation policy) developed with application of other models.

The stochastic simulation model was the main tool applied for analysis of the water resource system in the Odra River Basin and for the design of its expansion. Therefore, it is discussed in greater detail in Section 4.

3.4 Maximum Release Model

This model was used by Kos (1979). The results concerning the draft were very close to that of the chance-constrained Model I (for maximum draft and direct solution). However, the flood control objective was met with higher reliability in this model. Therefore, the conclusion that this model is preferable to the other model compared is obvious. The problem is, however, in determining the operation policy in keeping with this model which is practicable. The reservoirs in the Odra River catchment are situated in the upper part of the river catchments. Therefore, they control a smaller part of the drainage basin. On the basis of the flood damages that could be prevented by reservoirs, the relatively lower degree of the importance of flood control by reservoirs was determined. In the chance-constrained model, this lower degree can be reflected by lower reliability in the flood control constraint. The release maximum model gives the same importance to flood control and other purposes. Therefore, the boundary conditions it offered and the results were considered in the final decision analysis as information concerning the limits.

3.5 Flows in Network Models

The water resource system of the Odra River was recently given new impetus for investigation. It was necessary to meet some new industrial demands before the enlargement of the system by the Slezská Harta reservoir. The system is interconnected by many industrial pipelines which ensure that two or three sources can be used in case of hydrological or operation failure. This network of pipelines can be used not only for factories and metallurgy but for the purposes of water resource system development too. The best way to reduce the probability of water supply failure until the reservoir Slezská Harta can be operated is studied by the method of flows in networks. Such tasks as the maximum permissible flow, the flow of minimum costs, etc., are investigated. The preliminary results are promising.

3.6 Application of Game Theory

This part of the investigation uses a different approach. The assumptions of the models mentioned can be fulfilled when the users act in accordance with the derived operation policy. However, in some cases, there is no legislative means to force the users to do this. The consequences of their real behavior were tested by application of the non-antagonist cooperative game and non-cooperative game theories. The results have shown that the behavior of some users will not have a substantial influence on the system, while others can lower the reliability of the system in critical periods.

Therefore, the attention of lawyers concerned with the water resource systems in question was focused on the second group of water users to come to an agreement for modification of the current water rights.

4. THE SYNERGISTIC MODEL

As the basic method for the design of the operation policy of the water resource system in the Odra River Basin synergism (for this term and method see Hirsch, Cohon and ReVelle 1977) with the stochastic simulation model was used. This method requires the least simplification, and investigation of the critical periods with deficits in water supply is possible. This method enables unification of the design of the system with operation policy determination.

4.1 Main Elements of the System

The main elements of the system consist of the reservoirs, the diversion canals, the water treatment stations, the network of main pipelines, the water demands, the flood control measures, and the recreation facilities. The principle data on the reservoirs are given in Table 2.

The subsystem of the public water supply of the city of Ostrava and its suburbs has the following reservoirs: Kružberk, Šance, Morávka and the planned reservoir Slezská Harta. The industrial demands are represented by

(a) metallurgy (water is supplied mainly from the reservoir Žermanice),
(b) the ironworks (the river flow regulation of the Ostravice River by the Šance reservoir and of the Olše by the reservoir Těrlicko),
(c) the thermal power stations,
(d) the water resource subsystem of mines.

Flood control storage in this model was considered as the constraint given by previous models.

The hydrologic input was formed by the monthly synthetic flows for 500 years in the system of gauging stations. The geographical conditions of the system required a special combination of synthetic flows generation. Approximately one half of the system is situated in the area of the Jeseníky Mountains and the rest in the area of the Beskydy Mountains. These two mountain ranges have different geological structures and different exposure to the direction of wind. The hydrologic regime of these two catchments is different too. Therefore, the monthly data in the two stations were generated by the method of principal component analysis (i.e., for Kružberk in the Jeseníky Mountains and Šance in the Beskydy Mountains) the other monthly flows were generated by the method of central and satellite stations (see Fiering 1962). The three-parameter logarithmic transformation of the monthly flows was used. The additive parameter in this transformation was carefully analyzed as it has a decisive influence on the operating procedure (Kos 1969).

The Ostrava's public water supply subsystem is such that it is technically possible to provide for cooperation among the reservoirs (connection by main pipelines, the arrangement of the reservoirs, etc.). The isolated design of reservoirs for the municipalities is often done for a constant draft. In the water resources system, the reservoirs can be operated for a variable draft determined in such a way as to meet the constant total demand for the whole water supply subsystem. The principle behind this method was called synergism by Hirsch, Cohon and ReVelle (1977).

The effect of synergism is dependent on the hydrologic regime of the reservoirs. These authors state a substantial gain from daily operation of three reservoirs but not for the monthly operation. The reservoirs in the water resource system of the Odra River drainage basin have different regimes as they are in two geographical units and because they have different relative storage (defined as a ratio of the active storage to the mean annual runoff). Therefore, an operation policy with the monthly flows was sought to maximize the effect of cooperation.

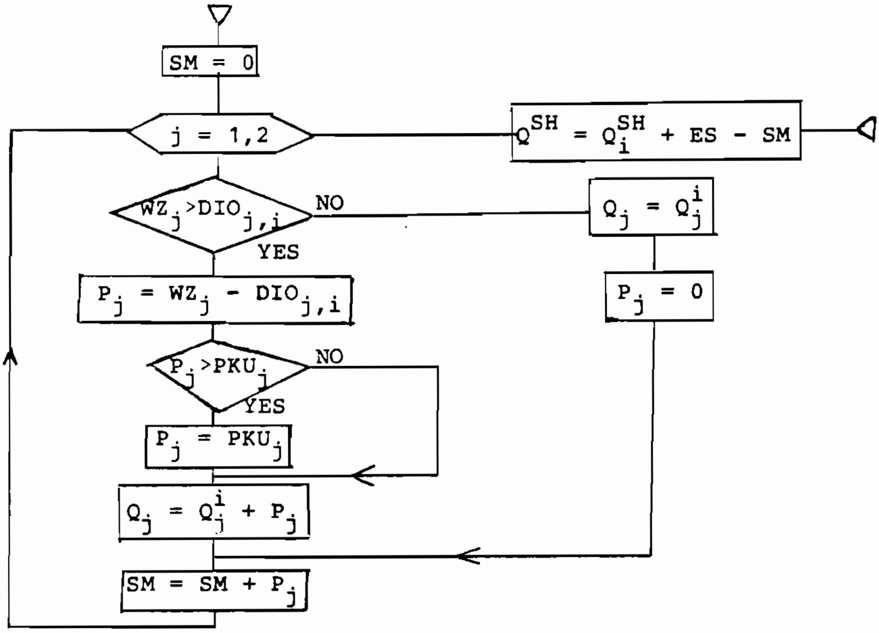
The operation policy is dependent on the hydrologic regime, the capacities of the main pipelines and on the capacities of the water treatment stations. The relation of hydrologic regimes can be characterized by the coefficient of correlation between the annual flows and corresponding monthly flows in the gauging stations compared. The less these coefficients are, the higher the effect will be due to the asynchronic hydrologic regimes.

The first step for determining operation policy is the division of the reservoirs into groups according to the length of their critical periods (release-filling periods). For instance, the reservoir Slezská Harta has a long-term carry-over potential (operating cycle or critical period 4-10 years), the reservoir Šance a 1-2 year critical period and the reservoir Morávka has a critical period of several months. Therefore, the reservoir Slezská Harta was classified as a long-term reservoir and the reservoirs Šance and Morávka as seasonal reservoirs.

During the operating cycle of the reservoir Slezská Harta, two or three operating cycles of the reservoir Šance can be closed, as well as several in the reservoir Morávka. Due to asynchronic hydrologic regimes of flows the critical periods do not coincide and therefore the draft of the reservoir with carry-over can be in some periods reduced and the seasonal reservoirs can be used up to the maximum technically and operationally feasible extent. This maximum is limited by the capacity of the treatment stations and mains. The total capacity of treatment stations should surpass the sum of the constant drafts. Otherwise cooperation would not be possible. The value of the difference between these two values is an economic problem. It was solved by the minimization of the total costs for the demanded draft.

The principles of synergism used in the operation policy are as follows: For each seasonal reservoir the rule curve has been determined taking the reliability and the draft that can be secured by the isolated function of the reservoir as input values. These rule curves divide the active storage of the reservoir into two parts. The lower part (under the rule curve) determines the storage that is necessary for the constant draft with a given reliability. The second part (above the rule curve) can be used for the cooperation of reservoirs in the system.

For the reservoir with a long-term operating cycle (Slezská Harta) the rule curve is not determined. Its operation policy is more complicated as it covers the rest of the system's demands. The principle of this operation (see Zeman 1978) is given by the flowchart in Figure 3.



i - index of the month

j - index of the reservoirs Morávka and Šance

WZ_j - storage of water at the beginning of the month

$DIO_{j,i}$ - storage under the rule curve

SM - the additional draft of reservoirs Morávka and Šance

P_j - the additional draft

PKU_j - the increment of water treatment capacity

Q_j - the draft in cooperation of reservoirs

Q_j^i - the isolated draft

Q^{SH} - the draft of the reservoir Slezská Harta in the system

Q_i^{SH} - the draft of the isolated reservoir Slezská Harta

ES - the effect of the system, i.e. the draft increment

Figure 3. Flowchart of reservoir cooperation.

The total draft of the system of reservoirs is equal to the sum of the drafts of isolated reservoirs, plus the increment of the draft of the reservoir Slezska Hartá, i.e. the reservoir with the long-term operating cycle. This increment is the effect of the operation policy in the system. The value of this effect was approximately 0.5 m^3/s, that is approximately 7% of the total draft. The sum of the isolated drafts was approximately 7.3 m^3/s, i.e. 2.0 m^3/s from Šance reservoir, 0.5 m^3/s from Morávka reservoir and 4.8 m^3/s from the cascade Slezská Harta-Kružberk. The total draft in the system was 7.8 m^3/s.

In designing the operation policy of the cascade (reservoirs in series) Slezská Harta-Kružberk the primary aim was the delivery of water for public water supply and the secondary aim was flood control, power production, recreation, etc. For fulfilling the primary aim, the water quality problem should be solved. Therefore water for public water supply has to be delivered from the reservoir Kružberk with the following operation policy: The releases should be from the reservoir Slezská Harta till its active storage is completely used, and then they should be effected from the reservoir Kružberk. Then the water level in the reservoir Kružberk has to be kept constant as long as possible and water can be withdrawn from the deeper layers of the best water quality.

The determination of the rule curves for the reservoirs Šance and Morávka was dependent on the demands of industry for the river flow regulation downstream of these reservoirs. Nevertheless, the rule curves were determined (by a heuristic approach with the knowledge of the current operation) for higher reliability of the water supply to the public (97-99%) and lower for industry (95-97%).

In this model the reservoir Těrlicko makes up part of the second subsystem where the delivery of water for industry and recreation were considered the prior aims. This determination will be possible in the water resource system expanded by the reservoir Slezská Harta where the need to maintain the water level for recreational purposes will be higher than in the current system. The multipurpose aspect of the system will thus be strengthened by the Slezská Harta reservoir's operation.

The multipurpose objectives used in the design of the reservoir Slezská Harta took into account purposes other than those mentioned. These purposes were expressed by the following criteria: the marginal costs of one unit of the draft, the total costs, the total draft, power production, quality of water entering the treatment station under the reservoir Kružberk, flood control, the influence of the reservoir on the environment, the reliability of the draft and operation till the reservoir is filled for the first time, conditions for the further development of the water resources system, the attitude of policy makers, construction problems (geological conditions) and the number of people to be relocated.

The design of the optimal range of the reservoir capacity was done by the method of decision analysis with the application of Fuller's method, independently by ten experts. Then the results were summarized and processed by the Delphi method.

The result of this investigation was the non-inferior range of the reservoir capacity (190-230 million m^3). For planning purposes, the mid-point (210 million m^3) of the range was determined as the resulting value.

The water resource system of the Odra River catchment is an example of the practical application of the systems approach to water resources development, with the utilization of operations research and simulation models in combination.

5. CONCLUSION

The investigation of the water resource system of the Odra River catchment has proved the need for a link between the theoretical approach and applied systems analysis. In models, reality is necessarily simplified: therefore, theoretical models of water resource systems have some drawbacks and some advantages.

These are revealed during practical application. Operation policy determination then, is the crucial point of the art and science of the design of water resource systems. The limitations and constraints of model application demand that the model be tailored to the specific case and the modification of general methods is often necessary.

A number of methods have been developed for the design of multipurpose water resource systems and policy determination. However, these individual methods do not offer a comprehensive solution to the problem.

In the water resource system of the Odra River basin, the method of multimodelling was an attempt to apply the systems approach. The different models, namely a combination of the stochastic simulation model with synergism and the chance-constrained model, the model of flows in networks and the theory of games with subsequent verification by the simulation model, were the most suitable methods. These offered the best results for a practical operation of the system.

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TECHNOLOGY TRANSFER IN RESERVOIR SYSTEMS OPERATIONS

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SUMMARY

Technology transfer, i.e. the rate at which research results are tested, adapted, and adopted in practice, is discussed with special reference to the use of systems analysis in the operation of multiple reservoir systems. Following comments on the difficulties of technology transfer, a particular approach towards stimulating it is discussed, namely an especially designed workshop. The structure of this workshop is outlined.

I CONTEXT

Systems analysis, a new field of study based on a synthesis paradigm, did start its explosive growth nearly two decades ago. This growth was brought about by the confluence of several important developments. Among these were, and for that matter still are: (a) concern about environmental resource depletion and/or degradation (representing a major social development); and (b) the rapid evolution of computers (representing the most influential technological development of this century) together with their frequent use in management and control.

As an intersection of these two developments, "systems analysis in water resource management" has been a widely supported field of study from the outset. However, the fruits of this support, i.e. the return on research investments, remain very difficult to identify. Recently this low return has been called a problem in "technology transfer." For our purposes we will define this somewhat ill-chosen terminology as follows: technology transfer is the rate at which research results are tested, adapted, and adopted in practice. Both the manifestations and the nature of the technology transfer problem associated with the use of systems analysis in water resource management will be discussed briefly. This will be followed by a description of one of the efforts that are underway to contribute something to the problem's resolution.

II ILLUSTRATION

Sensitivity to the problem of technology transfer on the part of the writer dates back to the mid-sixties when completing an operations study for a system of four reservoirs. Three of these reservoirs were under construction and the fully designed fourth one, located at the edge of a city, was in various stages of being approved when it became a source of conflict. A coalition of environmentalists (decrying the "rape" of natural rivers), of farmers (fearing less than full compensation for their land), and conservative city dwellers (opposed to further increases in planning and in amenities paid from taxes) battled a coalition of recreationists (the city had no outdour recreation within 100 km), land developers (visualizing new housing around the reservoir), and business leaders (hoping for "growth").

The writer offered to build a simulation model to show any interested party the flood control, water supply, and recreation effects by having or not having the fourth reservoir. In addition, a search was made for the optimal operating rules for both the three reservoir and the four reservoir system. The purpose of the study was to "produce a superior arrangement of information" that could aid "the decision making process." Today such a study is called a "systems analysis."

Despite fairly adequate circulation of study results including a radio inverview and the publication of pretty graphics of reservoir fillings and river stages, the study results had zero effect on the final decision. Simplistic, often irrelevant arguments were used in a fight that environmentalists won. At best some coalition members of both sides pulled an incidental study result out of context and used it to support already adopted positions. This illustrates that an implicit assumption of many systems analysts, namely that their well-studied information will increase the rationality of decisions, can be questioned.

Was a real-life political decision making process the right depository for systems analysis results? Would it have been more appropriate to only share the study result with the Government Agency that had proposed and would actually operate the above reservoir system? Certainly fellow-engineers of the agency involved in reservoir planning and operations would be more receptive and find the study of some aid in their work.

As it turned out, the study results and novel methodology reports went right to the agency's bookshelf and had no effect on their operations practices. Was then the computer simulation model too simple and therefore not useful to the reservoir operators? Probably, although this was never tested. Were the operations procedures in use already close to optimal? Perhaps, although a proof was never sought. Assuming that near-optimality exists in reservoir operations, should one not direct systems analysis at the truly complex problems and seek to couple engineering models to sociological and ecological ones? It is likely that this will decrease technology transfer. The exhaustive SCOPE study[1] concluded that decentralization in decision making causes that only at the operations level (where the focus is on "single-purpose use of a single resource") modeling is a major management tool. This brings us back to engineering and engineering-economy models as the more promising area of systems analysis and as a fruitful area to study for the presence or absence of sufficient technology transfer.

III PROBLEM MANIFESTATION

Water systems management and planning research would seem to be a healthy enough member of the systems analysis research family. The numbers of related research projects, committees, meetings, publications, and participating disciplines are still growing. Indications are, however, that a mismatch has developed in the rate at which research results are produced and the rate at which they can be absorbed in practice. Among those indicators are:

(a) studies and symposia whose only concrete result is the recommendation to have more of the same;

(b) repeated calls for systems analysis case studies;

(c) increased production of models beyond any reasonable availability or, more seriously, beyond possible measurability of required data;

(d) models whose use would be many times more expensive than the practical solutions in current use;

(e) publication of models that are unvalidated, untested, and barely related to real world water resource management;

(f) continued entry of researchers who are unfamiliar with water systems features and/or water resource planning and management practices.

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[1] *SCOPE (Scientific Comm. on Problems of the Environment): "Environmental Modeling and Decision Making, the U.S. Experience," Holcomb Research Institute; Praeger Press (1976)*

IV NATURE OF THE PROBLEM

One way to characterize the situation is by stating that the research supply exceeds the demand. Reasons for the apparent poor coupling of supply and demand are not difficult to identify. Established research agencies rather than users tend to pay for water management research. Such agencies commit funding to multi-year goals. Introducing a corollary to Parkinson's law that "work will fill all available time", namely the rule that "research will use all available funds", the stage is set for overproduction of certain types of research. Furthermore, and more specifically: without well-designed constraints on project objectives (academic) researchers are liable to disregard the purposive nature of systems analysis. References to "decision makers" and the "decision making process" become mere code words to legitimize the pursuit of what is both relevant and least time-consuming to the investigator. Of paramount interest to many researchers is the perpetuation of their particular studies through a continuous cycle of needing new data to test models or methodology and needing new models or methodology to describe the collected data. To be sure, this is well within the scientific tradition which asserts that science is entitled to its internal goal, namely the unfettered pursuit of science. Whether this goal should ever be central to systems analysis may be questioned seriously.

Rephrasing one may say that new fields of research (e.g. water systems management), given continued funding, will tend towards a selection of internal goals. They grow in a feedback fashion with current research defining new problems and new problems stimulating more research. This growth continues until funding levels for the type of research in question are reached. With reference to reservoir systems management, this feedback is represented symbolically in Figure 1 by the $1 \rightleftarrows 2$ relation and by the resulting cycle I.

Turning to the demand side (i.e. to the agencies that manage reservoir systems) a similar tendency towards self-sufficiency and the selection of internal goals is found. Operating staffs will adjust initially designed reservoir operating rules in response to what they learn about: (a) the physical river system behavior (including changes induced by the reservoir and their release rules); and (b) the demands exerted by interest groups (owners of flood-prone land, recreationists, power users, irrigators, etc.). In time these changes become institutionalized leaving little in the way of policy space.

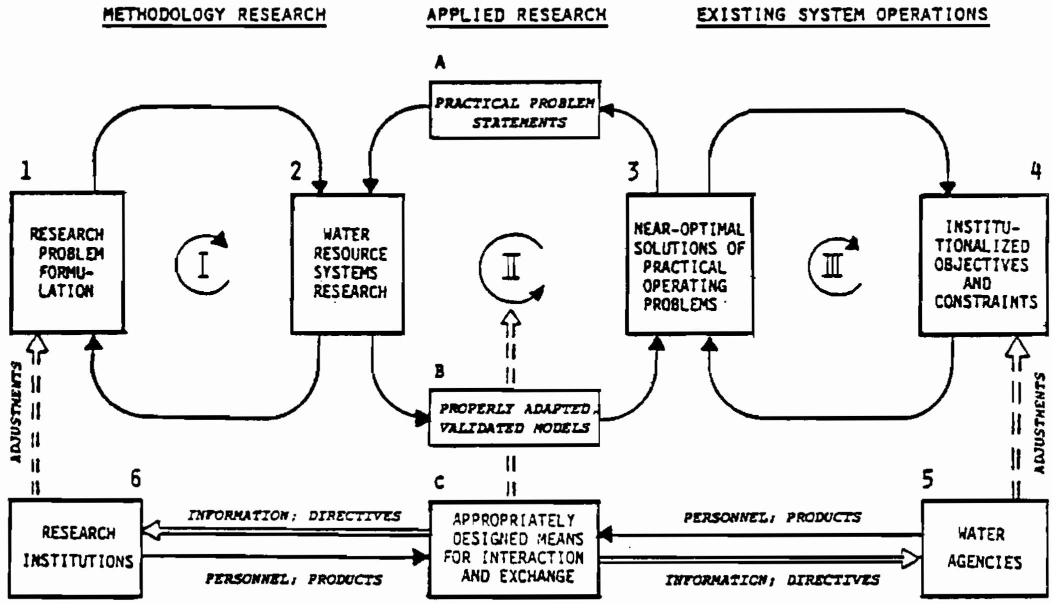

FIGURE 1 - CYCLES I AND III THAT TEND TOWARDS SELF-SUFFICIENCY AND A PURSUIT OF INTERNAL GOALS IMPEDE TECHNOLOGY TRANSFER, I.E. THE INFUSIONS A → 2 AND B → 3. APPROPRIATELY DESIGNED EFFORTS AND INSTITUTIONAL ADJUSTMENTS ARE THEN NEEDED TO STRENGTHEN THE CYCLE II.

Also budget and personnel are then reduced to match tasks that may become increasingly routine. This tendency towards self-sufficiency and a dominance of internal goals is represented in Figure 1 by the 3 $\rightleftarrows$ 4 relation forming a self-sustaining cycle III. Its persistence derives from the understandable pursuit of agency preservation objectives as well as the institutionalization of reservoir operations rules.

Without measures to stimulate researcher-practitioner dialog or collaboration the normal lack of communication tends to persist and with it the problems of low effectiveness of research expenditures and, at the same time, a persistence of antiquated operating practices.

V SOLUTION APPROACHES

In order to couple supply (of reservoir systems research) and demand (for improved field operating procedures) two types of exchange are needed, namely: (A) sufficiently generalized practical problem statements, and (B) models that are properly validated and adapted. Achievement of either one requires collaboration between practitioners and researchers. Symbolically the coupling may be represented by the cycle II, i.e. 2 → B → 3 → A → 2 in Figure 1.

Unfortunately the relation 2 $\rightleftarrows$ 3 is not a natural interaction so that the applied research cycle II is not self-sustaining. The sharing of detailed operational experience to achieve the transfer of practical problem statements (3 → A → 2, Fig. 1) represents a time consuming communications problem that competes for resources with the internal tasks of both the operating agencies and the research community. Furthermore, from a bureaucratic point of view a practitioner may have little motivation to share information that may lead to a critique of routine practices he designed to minimize critique and change. Likewise researchers have little interest in investing time in obtaining information that may hopelessly complicate their models or that might show interesting work on methodology to be irrelevant in practice.

Similarly, researchers tend to see the validating and adapting of models (2 → B → 3, Fig. 1) as a seemingly less productive effort in that it does not lend itself to generalization and can require substantial but routine data management efforts. And from the practitioner's point of view adapting new methods requires a considerable effort to obtain needed resources while running the risk that the new models will represent little or no improvement. Especially troublesome can be that proposed methods tend to disregard legal and administrative givens. Their formulators often assume conditions whose establishment would demand revolutionary changes in established routines rather than more acceptable incremental, i.e. bureaucratic adjustments.

Clearly it requires special effots to generate and sustain technology transfer, symbolically represented as the cycle II in Figure 1. Among the available approaches are:

(a) publish and read more reports and papers on reservoir operations,

(b) organize seminars, short courses, and workshops on reservoir operations,
(c) create a systems research group within the water agency,
(d) purchase applied research from an outside research group,
(e) have research funding agencies include technology transfer as a required component of water management research proposals,
(f) exchange personnel between research organizations and water agencies.

Each approach has its strengths and weaknesses, its time constant as well as appropriate timing. No single approach is best under all conditions. Rather some program that rotates and integrates elements of these approaches is likely to be optimal. Even then much depends on competence and leadership qualities of those involved.

In considering a program to increase technology transfer in the area of reservoir systems operations a fundamental difficulty should be kept in mind. This fundamental problem discussed above is that institutionalized objectives (1 and 4, Figure 1) effect a closing of the cycles I (research) and III (practice) at the expense of the cycle II (technology transfer). Consequently it will be necessary to achieve adjustment or a broadening of objectives (6 → 1; 5 → 4) so as to include contributions to technology transfer among its goals. Now a change in objectives does require substantial motivation, commitments, and the exercise of influence. For this reason the writer concluded (in 1976) that the most effective approach would be to bring together the leadership in reservoir systems research and in the operations branches of principal water agencies for a structured, intensive exchange of views. The most feasible vehicle seemed to be a workship. Proposals to organize such a workshop found acceptance (IIASA Workshop on Operation of Multiple Reservoir Systems, Jodlowy Dwor, Poland, May 28-June 1, 1979; J. Kindler, Chrm.; ASCE-OWRT Workshop on Reservoir Systems operations, Boulder, CO, USA, Aug. 14-17, 1979; G. H. Toebes, Chrm.). At this stage only a few comments are possible on the structure and preparations for the Boulder conference, since it is yet to take place.

VI ORGANIZATION OF BOULDER WORKSHOP

The majority of the practitioners and of the researchers involved in reservoir operations share a civil engineering background. It was logical therefore to work towards a workshop via the well-developed technical committee structure of the American Society of Civil Engineers. Definite plans for a "National Workshop on Reservoir Systems Operations" were made at a 1977 meeting of the Water Resources Systems Committee of the ASCE Water Resources Planning and Management Division.

The immediate objective of the National Workshop is to bring together analysts and practitioners of reservoir systems operations for an intensive, carefully structured, participatory exchange of information and views. Long-range objectives included:

(a) increase researchers' knowledge of reservoir systems features, of operating procedures in current use, and of the institutional constraints on operational changes.

(b) provide practitioners with an opportunity to compare their operations practices with those of other reservoir systems with a view towards assessing the utility of newer analysis methodologies.

(c) promote the best possible adaptation of reservoir operating procedures to implement present-day and anticipated future water management goals.

As to the organizational approach, a 25-member Task Committee was formed composed of the potential direct contributors to the National Workshop. During its first meeting the Task Committee refined the original organizational suggestions, to read as follows:

(a) the workshop should be organized around selected case studies.

(b) sufficient case study detail should be presented in the form of technical exhibits bearing on current operations practices.

(c) multiple views should be developed for each selected case study, namely an administrator's, and analyst's, and a user's view.

(d) participants of the by invitation only workshop would all be expected to actively contribute to the analysis, the debate, and a synthesis of views of the case studies and the broader issues they will represent.

(e) several tutorials would be used to ensure that all who attended could share some basic systems concepts and mathematical programming terminology.

In view of the somewhat novel format of the Workshop it was decided that part of the second meeting of the 25-member Task Committee would be given to a trial presentation of one of the major case studies as well as to two of the didactic tutorials. This proved useful in somewhat adjusting the approaches of the Workshop presentations which each of the Task Committee's members planned to give. It confirmed the wisdom of attempting to form kaleidoscopic descriptions (using presentations by the analyst, the user, and the administrator) of the case study systems.

The two Task Committee meetings proved essential in arriving at commitments to contribute to the Workshop. Helpful in this regard was a 4-member Overview Committee composed of a high level administrator from each of the three major water management agencies providing the case studies and one representative of the research community known for his effective commitment to technology transfer efforts.

Another essential ingredient in arranging the Workshop was financial support. The Task Committee chairman prepared a Workshop proposal for the Office of Water Research and Technology, a national research funding agency for water resources research. It was funded and permits selecting the Workshop's participants who attend by invitation only.

MONDAY AUGUST 13 *DAY 0*	TUESDAY AUGUST 14 *DAY 1*	WEDNESDAY AUGUST 15 *DAY 2*	THURSDAY AUGUST 16 *DAY 3*	FRIDAY AUGUST 17 *DAY 4*
Exhibitors to arrange exhibits in place; installation and testing of terminals	Registration, Opening W.S. Tutorial A Tutorial B Tutorial C Tutorial D	Case A-1 Case B-1	Case A-3 Case B-3	Plenary Session
-	-	Selleck	-	-
Exhibitors to arrange exhibits in place	Tutorial E Tutorial F Study Meetings	Case A-2 Case B-2	Inspect Exhibits; Study Meetings	Meeting Editorial Committee
-	-	-	White	-
Registration	Inspect Exhibits	Social Program	Report Writing	

FIGURE 2 - LAYOUT OF BOULDER WORKSHOP ON RESERVOIR SYSTEMS OPERATIONS

A schematic layout of the Boulder Workshop is given in Figure 2. Morning, afternoon, as well as evening sessions are being planned. There will be three "major" case studies (A-1: Central Velley Project; A-2: Arkansas Project; A-3: TVA System) and three "minor" case studies (B-1: Columbia System; B-2: Lower Colorado System; B-3: Duke Power Company System. The tutorials cover: A - Systems Analysis history and terminology; B - Classification of Reservoir Systems; C - Data for Reservoir Operations; D - mathematical programming; E - history of systems analysis in water resource management; F - uses of systems analysis in water resource management. The exhibits will support nine working or study groups (2 groups per A-study; 1 group per B-study) in their debate of the case studies. The working group recommendations in the area of technology transfer will be formulated during study meetings and report writing sessions. Finally these will be heard and, hopefully, synthesized by a panel during a plenary session.

If letters from invitees whose schedules do not permit attending the Boulder Workshop are any indication, the selected format and topics are meeting with general approval and are generating a definite expectation that they will contribute to technology transfer in the area of systems analysis and operation of multiple reservoir systems.

OPERATION OF MULTIPLE RESERVOIR SYSTEMS (SUMMARY AND CONCLUSIONS)

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The purpose of this workshop was to present and evaluate the methodological background for real-time operation of multiple reservoir systems. Seventeen papers, including a number of case studies, were presented during the workshop, illuminating experience gained in 13 countries in the field of multiple reservoir management.

The general impression derived from the workshop presentations and discussions was that there is no sufficiently clear systematic approach to methodological problems associated with the operation of multiple reservoir systems. This situation can create difficulties in mutual understanding, even between people working on similar problems. The main methodological questions linked with the practice of multireservoir management can be eventually grouped in the following way:

(1) methodology of water resources development planning;

(2) development of operating rules (or guidelines) aimed at the long-term operation of multiple reservoir systems;

(3) elaboration of methods for real-time (in some cases on-line) operation of such systems.

The majority of the papers presented at the workshop were devoted to the last two topics. Some authors, however, also touched on methodological problems of water resources development planning.

The discussions at the workshop were organized around a number of questions formulated by the Organizing Committee, on the basis of participants' proposals, including the following problems:

- What is the nature of interaction between system analysts and decision makers in a multiple reservoir system?
- Institutional aspects of decision-making processes in a multiple reservoir system;
- What kind of methods, models and other techniques may be used for reasonable operation of planned or existing multiple reservoir systems?
- Research needs and the role of control theory methods in the operation of multireservoir systems;
- Methodology of definition, identification and quantification of water resource systems objectives.

A summary of the results of the discussions at the workshop is presented below.

Systems Analysts and Decision Makers in Water Resource Systems

There was general consensus that there is a need for closer interaction between water resource systems analysts and decision-makers. In most cases, multiple reservoir systems are extremely complex and very often there is no single decision-making body responsible for their operation. As in other cases, system analysts should provide the decision makers with methods and aids which enable them to make proper decisions on the basis of:

- Knowledge of the state of the system under consideration (hydrologic conditions, volume of water stored in reservoirs, water demands, etc.);
- Forecasted water resource supply and demand;
- Estimated consequences of possible decisions.

In many cases, the most useful aids for decision-making purposes are models or sets of models, which provide the decision makers (e.g., reservoir dispatchers) with some information concerning the future behavior of the system. However, the workshop participants were of the opinion that in most practical situations, and in particular, during critical operating periods such as droughts and floods, there is a need for close interaction between model builders and system operators. It was also stressed that in many cases, particularly for multipurpose water resource systems, decisions are usually based on subjective impressions and judgments. In such situations, models should simply provide timely and reliable information for decision making, e.g., information on the consequences that would result from the application of a set of possible control decisions, and in a format easily understood by the decision maker. It was also stressed that the evaluation of recent operating practices may lead to useful conclusions concerning future decisions and may help analysts develop models more acceptable to the decision makers.

Institutional Aspects

Several problems arose when the discussion turned to analyzing institutional aspects of the decision-making processes. Special attention was paid to the degree of decentralization necessary in decision making and operation. It was realized that there are usually several levels and types of decision-making authorities including water users, local or regional water authorities, and governmental bureaucrats or politicians. In many cases there are several organizations which have jurisdictional authority and responsibility in multipurpose, multi-reservoir systems. When these organizations act separately, it is usual to use a task force or committee approach to decision making, in particular during critical operation periods such as droughts or serious floods.

In cases when there are several institutions involved, but when an effective organizational structure does not exist, it may be necessary for system analysts to provide arguments and guidelines for demonstrating the advantages, possibilities, and effects of the institutional restructuring.

The problem of centralization or decentralization in the decision-making processes was not clarified sufficiently in the workshop papers. The conclusion was that the change to a more centralized decision-making structure could lead to a more efficient use of resources, but may not be costless or "optimal" in a more general sense. The question of automation in water resources management was also touched on during the discussion and it was generally agreed that complete automation is in most cases undesirable. In the case of extreme hydrologic events, manual monitoring and interventions in the systems' behavior is desirable and must be possible.

Methodology and Multireservoir Management

An extended discussion was held on methods and aids necessary for reasonable operation of multireservoir systems. It concentrated around the question: what types of modeling approaches are best suited to specific reservoir systems or operational problems. The answer to this question cannot be given in a straightforward fashion and many aspects of the modeling activities have been considered by the workshop participants.

The dominating impression was that the existing modeling techniques are still insufficient and that there is a need for improvement and development, both for planning and dispatching purposes.

One of the most interesting problems discussed focused on the type of operating rules used for operation of multireservoir systems. Two different approaches were suggested:

- o application of fixed (predetermined) operating rules to determine water releases from the reservoirs as a function of preselected state variables,
- o application of elastic methods of operation when optimization and/or simulation models are used during the decision-making procedure.

Most of the workshop participants expressed the opinion that application of fixed operating rules may only fail when the operation of multiple reservoir systems with many conflicting situations is considered. Another disadvantage of the fixed operating rules is that they work properly only in average operating conditions, but usually fail in extreme situations, such as droughts and floods. Some participants reported on a successful combination of both approaches in the following manner: application of elastic methods of operation (using optimization techniques) during extreme events, such as flood emergencies, etc., but leaving the predetermined operating rules only for short intervals, namely during the most critical time period. This technique which combines the advantages of both approaches seems to be an attractive solution.

Another extensively discussed problem is related to the applicability of optimization and simulation models in the operation of multireservoir systems. There was a general feeling that optimization techniques may be extremely useful when objectives can be quanitfied and when known optimization methodology can be applied credibly. There are, however, serious problems in proper application of such methodology in the case of large systems. This is in particular true for such well-known techniques as dynamic programming, which for many years was used to find the optimal operating rules for a single reservoir but because of the dimensionality problems, is of limited use for large multireservoir systems, even when decomposition strategies or aggregation methods are used.

It seems that for large water resource systems simulation techniques are of greater importance, in particular when there is a need to reflect very detailed aspects of the system's operation or where objectives of some of the water users cannot be quantified. Simulation models seem to be preferred if there is a need for closer collaboration between system analysts and decision makers.

There was general consensus that the combined use of simulation and optimization techniques may be fruitful both for planning and real-time operation purposes.

Some particular methodological problems were also raised by the workshop participants. There was a general feeling that the theory of operation of large hydropower systems is still at a relatively early stage of development and that there is substantial scope for developing more powerful optimization techniques to improve hydroenergy production. Special attention was focused on problems of systems operation during flood periods. It was stressed that there is a need for developing models which effectively combine meteorological observations and forecasting, runoff forecasting and the dynamic scheduling of reservoir releases. All these models should be reasonably simple, as the decisions have to be taken quickly and at frequent time intervals. In regard to meteorological information, the use of radar techniques seems to show some promise of being used efficiently in practice.

Research Needs

A considerable part of the discussion was devoted to research needs in the operation of multiple reservoir systems. The following topics seem to be of great importance:

(1) There is a potential major role for new methodological approaches based on the concepts of (a) hierarchical structures of water resource systems, (b) decomposition techniques, and (c) multi-level and multi-layer decision-making structures;

(2) there is a need for further development of optimization techniques and their applications both in long-term operation planning and in system's dispatch;

(3) more research activities are needed to develop improved methods for aggregation and disaggregation of multireservoir water resource systems; this work is closely related to the concept of hierarchical (multi-layer) control structures, where the proper simplification of the models and linkages between models are very important problems;

(4) more attention should be given to the use of control theory methods, assuming however that their effectiveness and efficiency in comparison with existing operational methods will be studied carefully before putting them into practice;

(5) although there are some differences of opinion about the potential benefits of further development of Moran's reservoir theory, some workshop participants stressed the need for an improved storage theory with the emphasis on multireservoir systems.

Identification of System Objectives

The last part of the discussions was devoted to problems associated with the definition, identification, quantification and interpretation of systems objectives. The general conclusion was that objectives, and accordingly the objective functions, may differ substantially, depending on whether the models are for planning or for dispatching purposes. As a rule they should be keyed into national and/or regional goals.

The workshop participants were divided in opinion on the treatment of multiple criteria in the objective function. It was agreed that in some cases it seems to be preferable to consider only economic objectives and to treat other goals as constraints in optimization models. In other situations however, it is desirable to construct a multicriteria objective function and to use appropriate solution procedures to identify the relevant tradeoffs among efficient solutions. There is a need for more fundamental research on how to identify, define and include multiple objectives in water resource systems analyses.

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